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May 1997

US Army Corps of Engineers
Waterways Experiment Station

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by Robert D. Carver, Brenda J. Wright

Approved For Public Release; Distribution Is Unlimited

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U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

Final report

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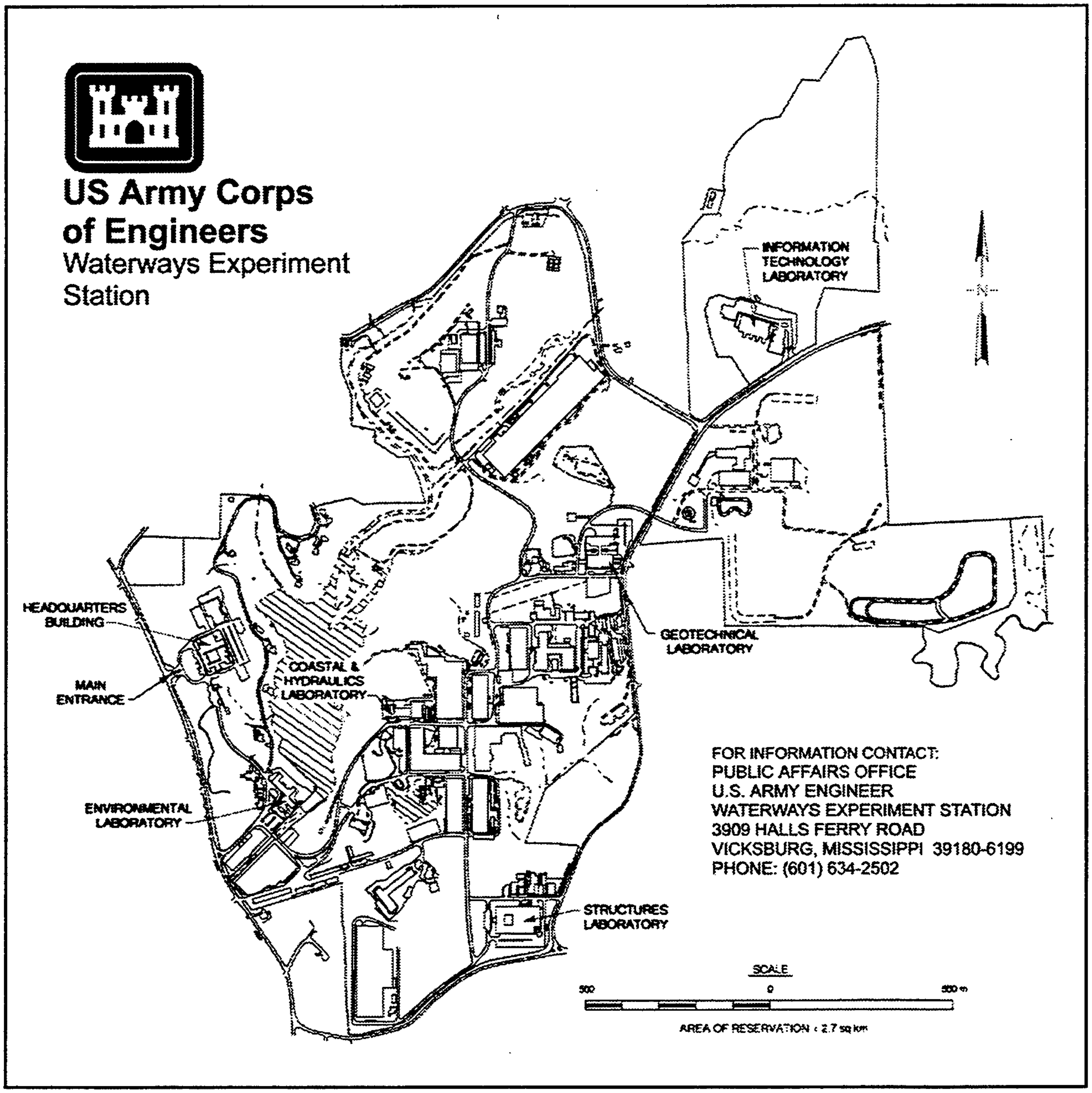
Prepared for Port of Long Beach
Long Beach, CA 90801-0570



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Contents

Preface

The model investigation described herein was requested by the city of Long Beach, CA, and was authorized by a Memorandum of Understanding between the city of Long Beach, acting by and through its Board of Harbor Commissioners, and the United States of America, represented by the Commander, U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, MS. Moffat & Nichol Engineers acted as overall project manager/designer for the Port of Long Beach and as such were responsible for determining design wave conditions, selecting breakwater sections for testing, and monitoring the direction of model tests.

During the investigation, Drs. Ying Poon and Kimo Walker and Mr. Russ Boudreau of Moffat & Nichol Engineers visited WES. Dr. Frederic Raichlin, consultant to the city of Long Beach, also visited the model and his suggestions are appreciated. Messrs. Mike Ellis and Steve Schryber of Connolly Pacific Construction visited WES and assisted in construction of the model test sections used in the low-density stone tests.

The study was conducted by personnel of the Coastal and Hydraulics Laboratory (CHL), WES, under the general direction of Dr. James R. Houston, Director, and Mr. Charles C. Calhoun, Jr., Assistant Director. Direct guidance was provided by Messrs. C. E. Chatham, Chief, Wave Dynamics Division, and D. Donald Davidson, Chief, Wave Research Branch, CHL. Tests were conducted by Mrs. Brenda J. Wright and Mr. Johnny Heggins, Engineering Technicians, under the direction of Mr. R. D. Carver, Principal Investigator, all of CHL. This report was prepared by Mrs. Wright and Mr. Carver. Word Processing and formatting were done by Ms. Myra E. Willis, CHL.

At the time this study was conducted Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	meters
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
miles (U.S. statute)	1.6093	kilometers
tons (2,000 pounds, mass)	907.1847	kilograms

1 Introduction

The Prototype

The Port of Long Beach is Located in San Pedro Bay, immediately east of the Port of Los Angeles (Figure 1). Presently, the harbor is protected by an 8-mile-long stone breakwater extending from Point Fermin eastward past Long Beach Harbor. The breakwater, consisting of three sections, was constructed in stages. First to be built was the San Pedro breakwater; later, the Middle breakwater was constructed; finally, the Long Beach breakwater was added.

Increased commercial activity has created the need for an additional rubble-mound breakwater to protect the Pier J basin. The newly proposed breakwater will be located on the leeward side of the middle breakwater and will provide for dissipation of long-period (40- to 200-sec) wave energy entering Queen's Gate and transmitted through and/or over the Middle breakwater.

Purpose of Model Investigation

The purpose of the investigation was to determine, by two-dimensional (2-D) flume tests, the stability response of the proposed breakwater as initially designed and, based on results of these tests, additional plans would be tested as needed. Also, transmission of long-period wave energy would be measured for selected plans.

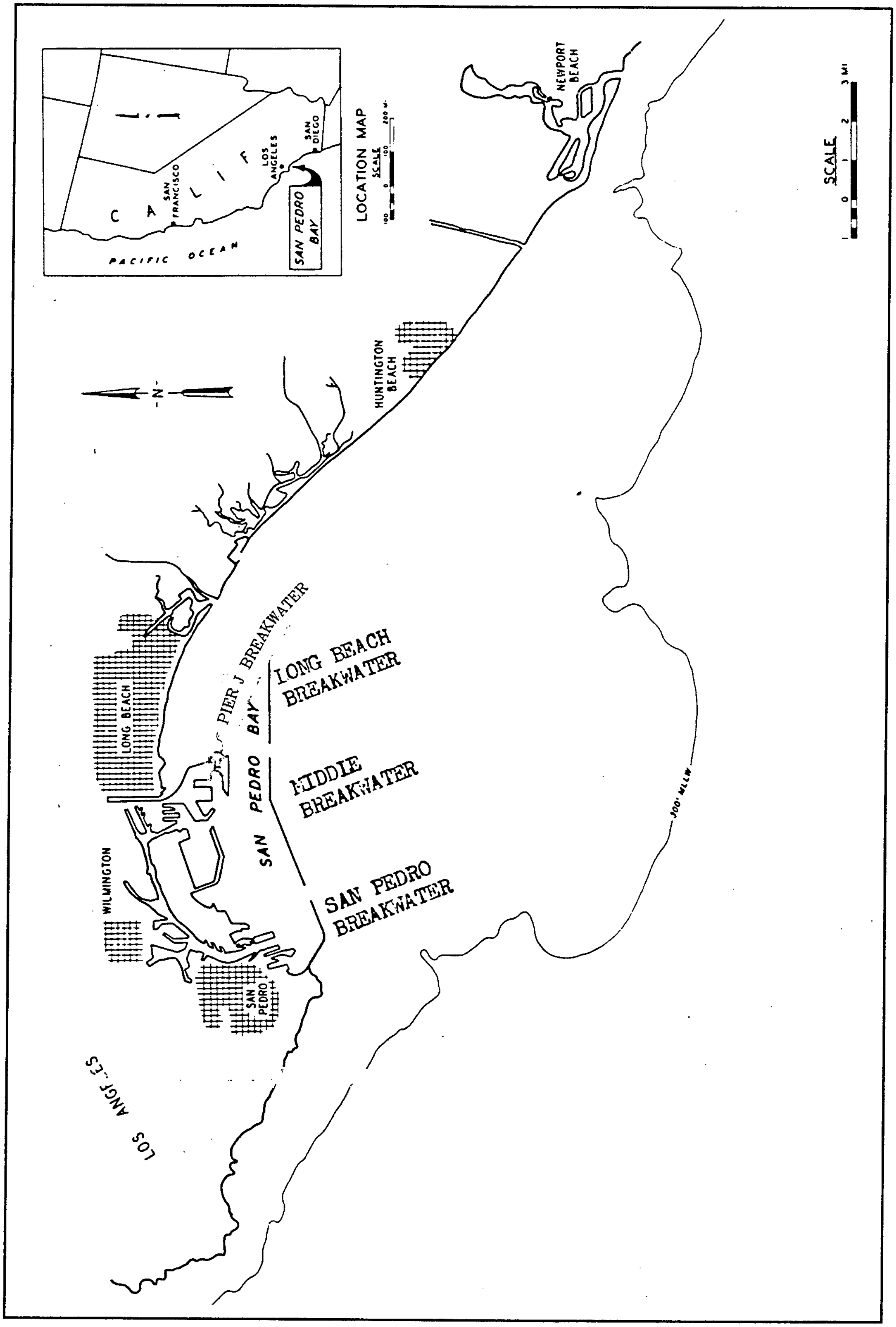

Figure 1. Site map

2 The Model

Model-Prototype Scale Relationships

Tests were conducted at a geometrically undistorted scale of 1:30, model to prototype. Scale selection was based on the sizes of model armor available compared with the estimated size of prototype armor required for stability, elimination of wave transmission scale effects, preclusion of stability scale effects (Hudson 1975), and capabilities of the available wave tank. Based on Froude's model law (Stevens 1942) and the linear scale of 1:30, the model-prototype relations in Table 1 were derived. Dimensions are in terms of length (L) and time (T).

Table 1
Model-Prototype Scale Relations

Characteristic	Dimension	Model-Prototype Scale Relation
Length	L	L_r = 1:30
Area	L^2	$A_r = L_r$ = 1:900
Volume	L^3	$V_r = L_r$ = 1:27000
Time	T	$T_r = L_r$ = 1:5.48

The specific weight of water used in model tests was assumed to be 62.4 pcf and that of seawater is 64.0 pcf. Also, specific weights of model breakwater construction materials were not the same as their prototype counterparts. These variables were related using the following transference equation:

$$\frac{(W_a)_m}{(W_a)_p} = \frac{(\gamma_a)_m}{(\gamma_a)_p} \left(\frac{L_m}{L_p}\right)^3 \left[\frac{(S_a)_p - 1}{(S_a)_m - 1}\right]^3 \qquad (1)$$

where

W_a = weight of an individual armor unit, pounds
m,p = model and prototype quantities, respectively
γ_a = specific weight of an individual armor unit, pounds per cubic foot
L_m/L_p = linear scale of the model
S_a = specific gravity of an individual armor unit relative to the water in which it was placed, i.e., $S_a = \gamma_a/\gamma_w$

In a hydraulic model investigation of this type, gravitational forces predominate (Froudian model law), except when energy transmission through the breakwater is considered (Keulegan 1973). If the core material was geometrically scaled according to Froudian model relationships, internal Reynolds numbers would be too low, and too much energy would be dissipated. Therefore, for all plans tested, the core stone was geometrically oversized according to Keulegan (1973) to aid in reproducing wave energy transmission.

Test Equipment and Facilities

All stability and undistorted transmission tests were conducted in a 3-ft-wide segment of a concrete wave flume 11 ft wide and 245 ft long (Figure 2). Distorted scale transmission tests were conducted in a 1-ft-wide segment of a concrete wave flume 3 ft wide and 150 ft long (Figure 3). Irregular waves were generated by hydraulically actuated piston-type wave machines in each flume.

Wave data were collected on electrical capacitance wave gauges which were calibrated daily with a computer-controlled procedure incorporating a least square fit of measurements at 11 steps. This averaging technique, using 21 voltage samples per gauge, minimizes the effects of slack in the gear drives and hysteresis in the sensors. Typical calibration errors are less than 1 percent of full scale for the capacitance wave gauges. Wave signal generation and data acquisition were controlled using a DEC MicroVax I computer. Wave data were analyzed using a DEC VAX 3600.

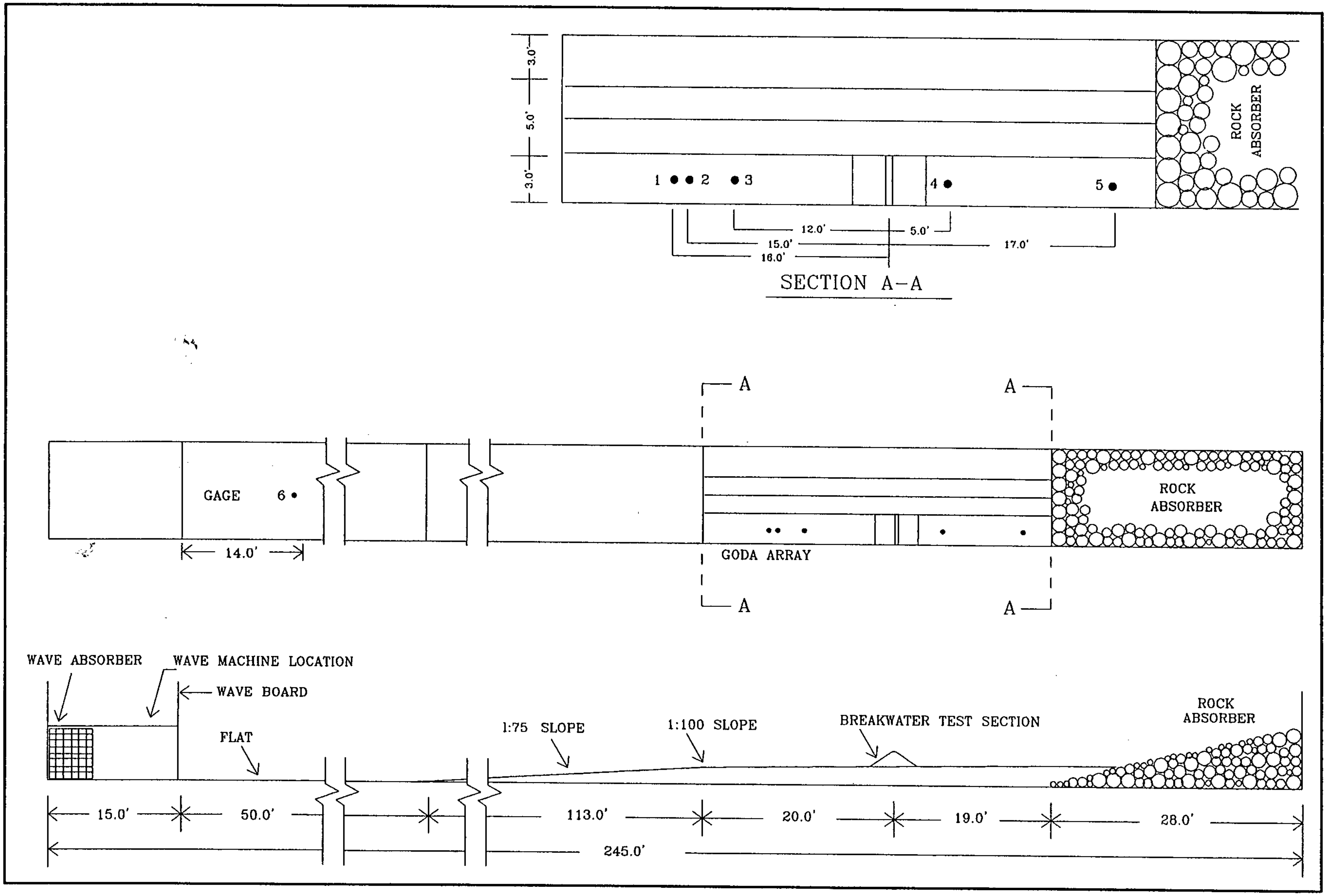

Figure 2. Wave tank cross section (stability and undistorted transmission tests)

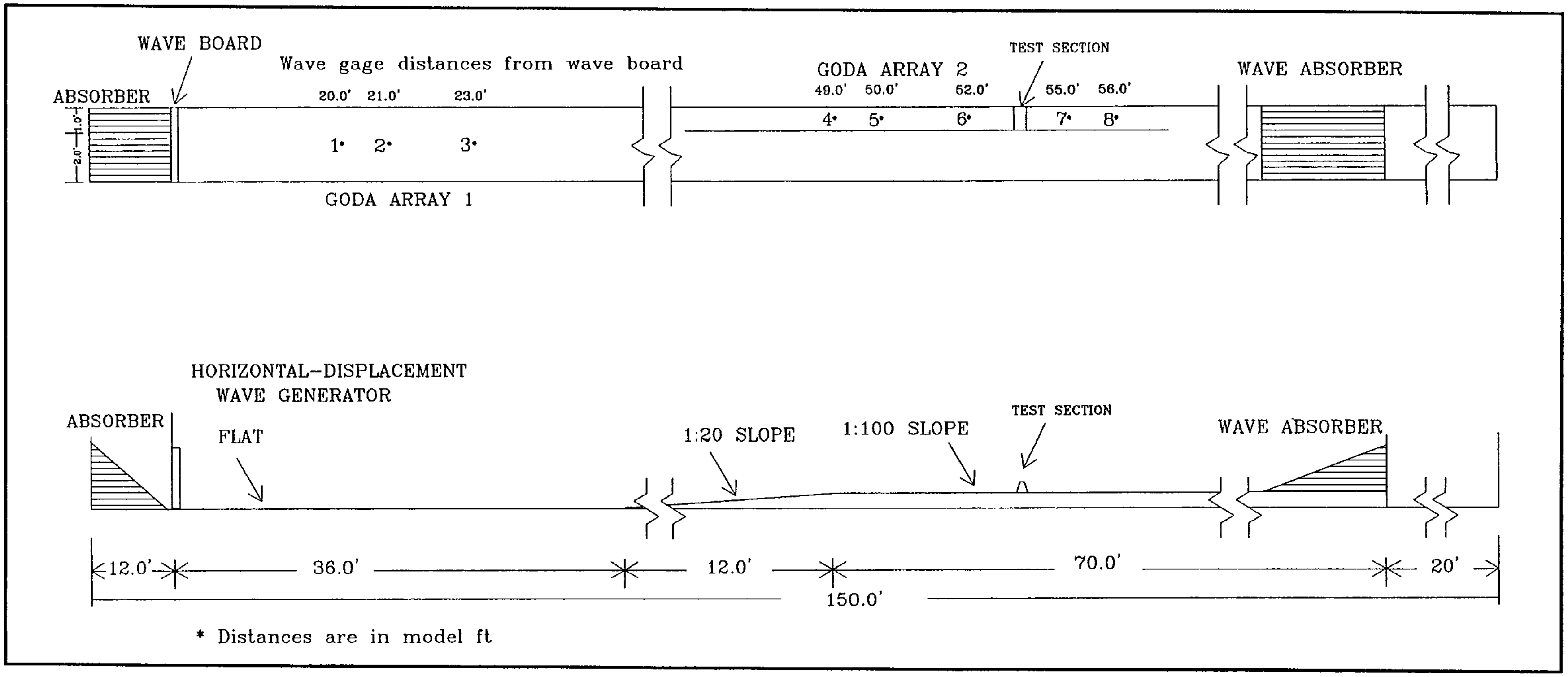

Figure 3. Wave tank cross section (distorted scale tests)

3 Stability Tests and Results

Method of Constructing Test Sections

All experimental breakwater sections were constructed to reproduce as closely as possible results of the usual methods of constructing full-scale breakwaters. The core material was dampened as it was dumped by bucket or shovel into the flume and was compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype structure. Once the core material was in place, it was sprayed with a low-velocity water hose to ensure adequate compaction of the material. The underlayer stone then was added by shovel and smoothed to grade by hand or with trowels. Armor units used in the cover layer were placed in a random manner below mean lower low water (mllw); i.e., they were individually placed but were laid down without special orientation or fitting. Special placement with the long axis of the irregularly shaped stones perpendicular to the breakwater slope was used above mllw. After each test, the armor units were removed, all of the underlayer stones were replaced to the grade of the original test section, and the armor was replaced. All model stone except Plans 3, 3R, and 3R1 had a specific weight of 165 pcf to match normal prototype specific weight ranging from 165 to 167 pcf. The Plan 3 test series used specially supplied model stone from Empire Quarry whose specific weight was 148 pcf.

Description of Plan 1

Plan 1 (Figure 4 and Photos 1-3) was constructed to a crown elevation of +16 ft mllw and used armor slopes of 1V:1.75H sea side and 1V:1.5H harbor side. A crown width of 14 ft, equivalent to three armor-stone diameters, was used. The structure was armored with two layers of 5- to 7-ton stone.

Selection of Test Conditions

Based on siting of the breakwater in deeper water, tests were conducted using irregular waves with a Joint North Sea Wave Project (JONSWAP) spectrum using a peak enhancement factor of 7.0. Peak wave periods of the spectra ranged from 12 to 18 sec. The proposed design wave height was 13 ft; however, the stability

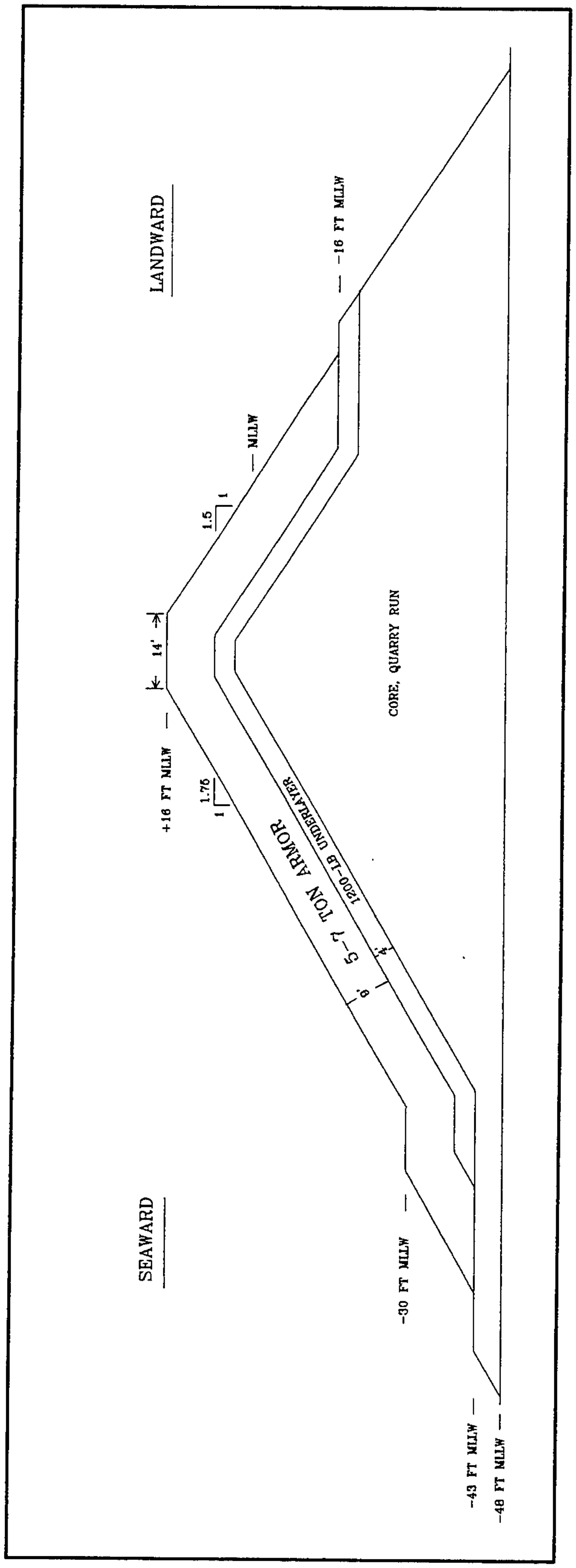

Figure 4. Characteristics of Plans 1 and 1R

response of each test section was investigated for up to 16-ft waves. Plan 1 was initially subjected to the storm hydrograph shown in Table 2 (Hydrograph 1):

Table 2
Test Conditions for Hydrograph 1

Step	SWL, ft mllw	T_p, sec[1]	H_{mo}, ft[2]	Duration, hr
1	+8.0	12.0	6.0	2.0
2	+8.0	14.0	8.0	3.0
3	+8.0	16.0	10.0	3.0
4	+8.0	18.0	12.0	12.0
5	+8.0	18.0	13.0	12.0
6	+8.0	18.0	16.0	12.0
7	0.0	18.0	13.0	12.0
8	0.0	18.0	16.0	12.0

[1] T_p = Wave period of peak energy density of spectrum, sec
[2] H_{mo} = Zero-moment wave height, ft

Transmitted wave heights were measured 150 and 200 ft shoreward of the breakwater. Goda and Suzuki's (1976) method was used to resolve the incident and reflected spectra.

Test Results of Plans 1 and 1R

Plan 1 exhibited an excellent stability response. Some minor reorientation of the armor was observed during steps 4 and 5. As shown in Photos 4-6, no displaced damage was evident at the conclusion of step 5. Step 6 displaced three seaward armor stones from the area between mllw and the crown. This number accounted for only 0.8 percent of the original armor, leaving the structure in excellent condition at the conclusion of step 6 (Photos 7 and 8).

Initially, it was planned to test only steps 1-6; however, the stability response of Plan 1 was so favorable that it was decided to assume that steps 5 and 6 could occur at an swl of 0.0 ft mllw. Thus steps 7 and 8, identical to steps 5 and 6 (except for the still-water level (swl), were created and added to the test. Step 7 initiated slight damage , 0.6 percent, on the landward side of the structure (Photo 9). No additional seaward armor damage was observed. Step 8 produced severe damage, 30 percent, on the harbor side as shown in Photo 10. Sea-side damage increased to 4 percent (Photo 11). The structure was considered failed at the conclusion of step 8.

Plan 1 was rebuilt and retested (Plan 1R) with Hydrograph 2, as summarized in Table 3.

Table 3 Test Conditions for Hydrograph 2				
Step	SWL, ft mllw	T_p, sec[1]	H_{mo}, ft[2]	Duration, hr
1	+8.0	12.0	6.0	2.0
2	+8.0	14.0	8.0	3.0
3	+8.0	16.0	10.0	3.0
4	+8.0	18.0	12.0	12.0
5	0.0	18.0	13.0	12.0
6	+8.0	18.0	13.0	12.0
7	+8.0	18.0	16.0	12.0
8	0.0	18.0	16.0	12.0

[1] T_p = Wave period of peak energy density of spectrum, sec
[2] H_{mo} = Zero-moment wave height, ft

As shown above, the first four steps of Hydrograph 2 were the same as Hydrograph 1. However, the 13-ft design wave initially was tested at an swl of 0 ft mllw followed by 13- and 16-ft waves at an swl of +8 ft mllw and concluding with 16-ft waves at the 0 swl.

Similar to the initial stability test, little movement was detected during the first four steps. During step 5, minor damage (2 percent) was incurred on the seaward face of the structure. The landward slope experienced major damage (15 percent). Photos 12 and 13 show the structure at the conclusion of step 5. Steps 6 and 7 produced little additional change except two more stones were displaced from the landward slope. As shown in Photos 14-16, step 8 produced a significant damage increase. At the conclusion of testing, the seaward armor from the area between mllw and the back of the crown had incurred 15 percent damage with stones being displaced both downslope, and over the crown. Much of the top layer of landward armor was displaced, resulting in about 40 percent damage. It is felt that the stability response in the repeat test was generally similar to the initial test, even though slightly higher damage values were observed.

Rationale and Description of Plan 2

Based on the stability response of Plan 1, it was decided to investigate alternative schemes that might improve stability or reduce the structure's cost without significantly affecting its functional performance. Some of the factors that govern material volumes and costs are elevation and width of the crown, type and weight of armor, and slope on which the armor is placed. It was decided that in this particular study the greatest cost savings with the least probable impact on functionality could probably be achieved by replacing the 5- to 7-ton toe armor with 1- to 3-ton stone (Photos 17-19 and Figure 5). Also, the 5- to 7-ton landward armor between the swl and -16 ft mllw was replaced with 8-to 10-ton stone in an effort to

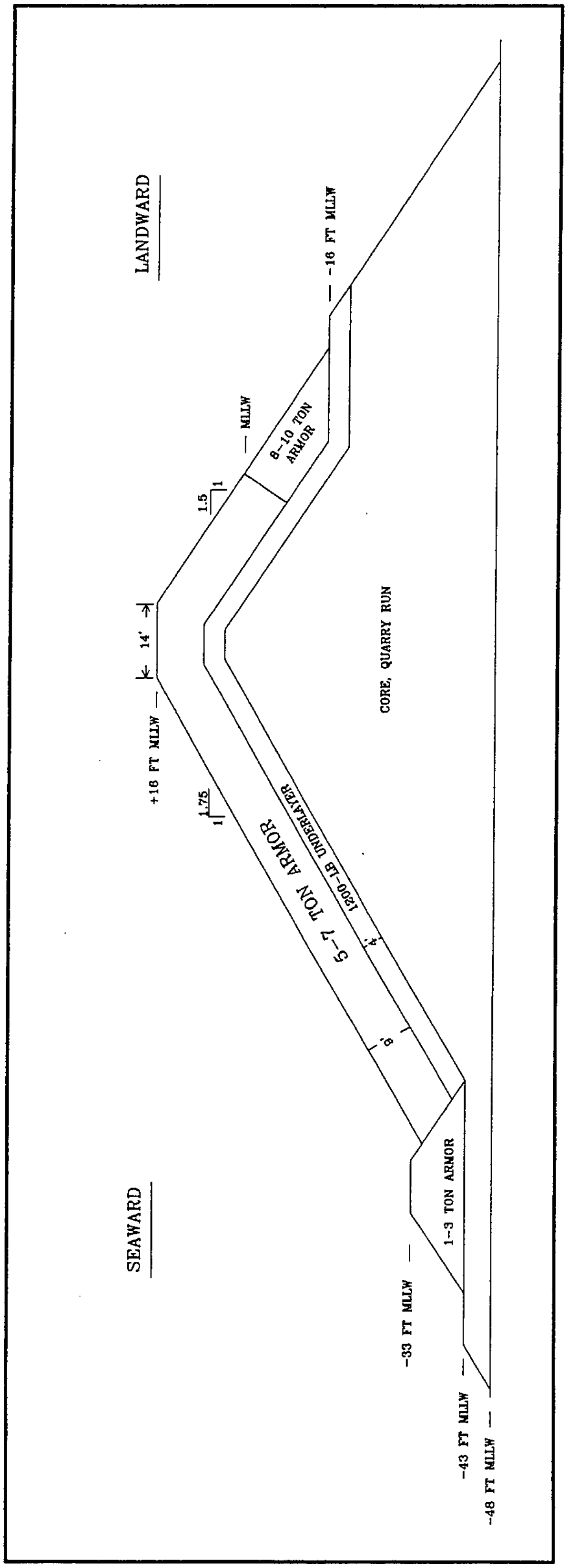

Figure 5. Characteristics of Plan 2

improve stability for the more extreme wave conditions. Also, it was decided to expand the hydrograph to include additional swls since initial stability response appeared sensitive to water level.

Plan 2 was subjected to the hydrograph shown in Table 4 (Hydrograph 3):

Table 4
Test Conditions for Hydrograph 3

Step	swl, ft mllw	T_p, sec[1]	H_{mo}, ft[2]	Duration, hr
1	+1.0	12.0	6.0	3.0
2	+1.0	14.0	8.0	3.0
3	-1.0	16.0	9.0	3.0
4	+1.0	16.0	9.0	3.0
5	+3.0	16.0	9.0	3.0
6	+8.0	16.0	9.0	3.0
7	-1.0	18.0	11.0	3.0
8	+1.0	18.0	11.0	3.0
9	+3.0	18.0	11.0	3.0
10	+8.0	18.0	11.0	3.0
11	-1.0	18.0	13.0	3.0
12	+1.0	18.0	13.0	3.0
13	+3.0	18.0	13.0	3.0
14	+8.0	18.0	13.0	3.0
15	-1.0	18.0	16.0	3.0
16	+1.0	18.0	16.0	3.0
17	+3.0	18.0	16.0	3.0
18	+8.0	18.0	16.0	3.0

[1] T_p = Wave period of peak energy density of spectrum, sec
[2] H_{mo} = Zero-moment wave height, ft

Test Results of Plan 2

No damage was evident at the conclusion of step 2. Step 3 displaced two seaward armor stones from the area between mllw and the crown. One additional armor stone was displaced from the same area during step 4. The three armor stones displaced accounted for only 0.7 percent of the original armor between mllw and the crown, leaving the structure in excellent condition at the conclusion

of step 4 (Photos 20 and 21). No additional damage was detected for the remaining 9-ft waves or the 11-ft waves. Step 11 initiated displacement of five additional seaward stones and two landward stones.

The breakwater was still in excellent condition at the conclusion of step 11, with 1.8 percent damage seaward and 0.4 percent damage landward. Steps 12-14 produced little change in the structure's condition. As expected, step 15 produced a large damage increase. Damage was estimated to be 12 percent seaward and 15 percent landward at the conclusion of step 15. Damage progressed during steps 16-18. At the conclusion of testing (Photos 22-24), the seaward armor from the area between mllw and the back of the crown had incurred 20 percent damage and landward damage had increased to 23 percent.

Rationale and Description of Plans 3, 3R, and 3R1

Another design parameter identified for testing was stone density. The Port of Long Beach was interested in considering stone of lighter density (148 pcft), which could be supplied from a local quarry at potentially reduced cost. Therefore, it was decided to develop an equivalent section with 148-pcf stone. Plan 3 (Figure 6 and Photos 25-27) was similar to Plan 2, except the 5- to 7-ton armor was replaced with 8- to 11-ton material and the 8- to 10-ton stone was replaced with 12- to 16-ton stone in an effort to achieve equivalent stability for the lower density stone. Also, the crest width was increased from 14 to 15 ft and underlayer weight and thickness were adjusted for the larger armor stone.

Test Results of Plan 3

Plan 3 initially was tested with Hydrograph 3. No damage was observed during the first 10 steps. Steps 11 and 12 each displaced one landward armor stone. No additional damage was detected for the remaining 13-ft waves. As anticipated, step 15 produced a large damage increase. Damage was estimated to be 10 percent seaward and 30 percent landward at the conclusion of step 15. Damage progressed during steps 16-18. At the conclusion of testing (Photos 28-30), the seaward armor from the area between mllw and the back of the crown had incurred 25 percent damage and landward damage had increased to about 75 percent with some exposure of core and underlayer near the crest.

Test Results of Plan 3R

Plan 3R was identical to Plan 3, except the model armor was placed by representatives of Connolly Pacific Construction Company. The resulting structure, shown in Photos 31-33, appeared similar to Plan 3, except the specially placed armor appeared even more uniform and the neat line was more closely met both below and above the water level. It should be noted that almost any level of uniformity can be achieved in the model and the previous plans were constructed to represent typical prototype results, based on the authors' observations of numerous prototype structures. Plan 3R also was tested with Hydrograph 3. No

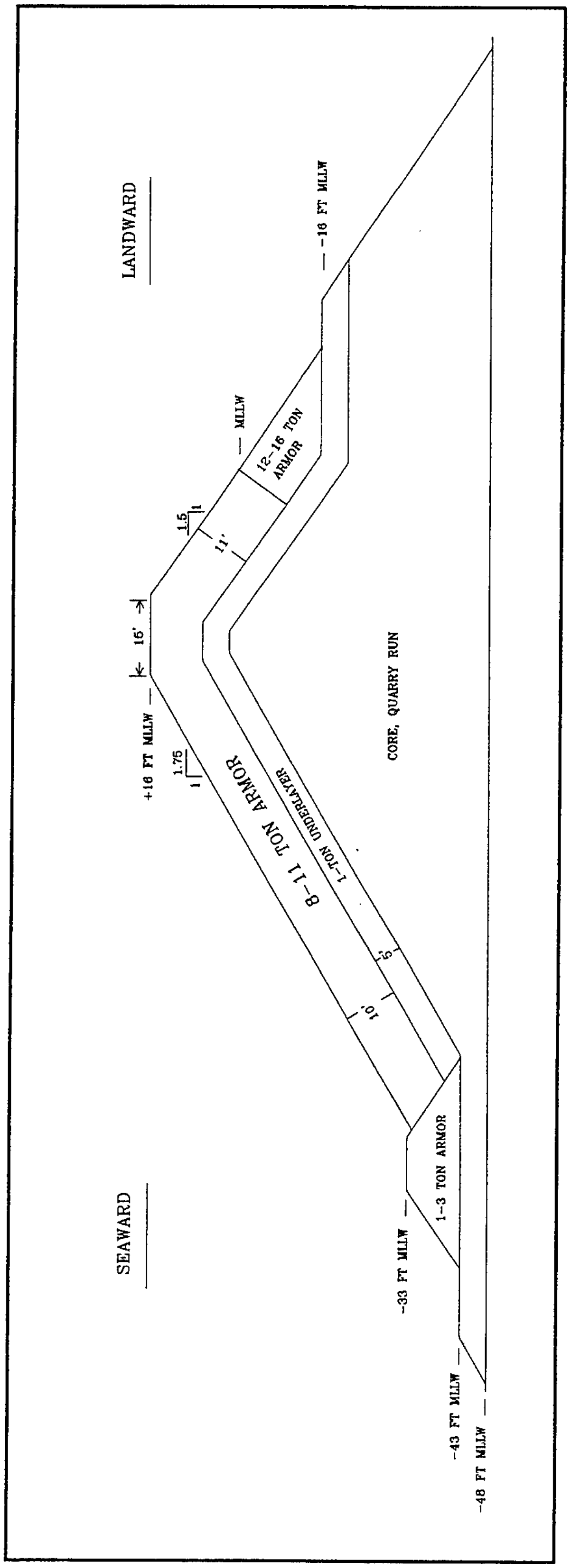

Figure 6. Characteristics of Plans 3, 3R, and 3R1

damage was observed during the first 14 steps. Two seaward armor stones and one landward armor stone were displaced during step 15 and one additional seaward armor stone was displaced during step 16. No additional damage was observed during steps 17 and 18, leaving the structure in excellent condition at the conclusion of testing (Photos 34-36).

Test Results of Plan 3R1

Plan 3R1 was identical to Plan 3. Since Plans 3 and 3R had yielded similar results for the 13-ft waves, but significantly different results for the 16-ft waves, it was decided to conduct a second repeat test. Plan 3R1 was tested with a shortened version of Hydrograph 3. Hydrograph 4, shown in Table 5, omitted the 6-, 8-, and 9-ft waves and the 11-ft waves at the +3- and +8-ft swls.

Table 5
Test Conditions for Hydrograph 4

Step	swl, ft mllw	T_p, sec[1]	H_{mo}, ft[2]	Duration, hr
1	-1.0	18.0	11.0	3.0
2	+1.0	18.0	11.0	3.0
3	-1.0	18.0	13.0	3.0
4	+1.0	18.0	13.0	3.0
5	+3.0	18.0	13.0	3.0
6	+8.0	18.0	13.0	3.0
7	-1.0	18.0	16.0	3.0
8	+1.0	18.0	16.0	3.0
9	+3.0	18.0	16.0	3.0
10	+8.0	18.0	16.0	3.0

[1] T_p = Wave period of peak energy density of spectrum, sec
[2] H_{mo} = Zero-moment wave height, ft

No damage was observed during the first 2 steps. One seaward armor stone and two landward armor stone were displaced during step 3. One additional seaward armor stone was displaced during step 4. No additional damage was observed during steps 5 and 6. Two additional armor stones were displaced from each side of the structure during step 7. No additional displaced damage was observed during steps 8-10, leaving the structure in excellent condition at the conclusion of testing (Photos 37-39). Thus Plans 3R and 3R1 both showed little damage (2 percent or less) after attack of 16-ft waves.

Discussion of Plans 3, 3R, and 3R1 Test Results

Are test results for Plans 3, 3R, and 3R1 fundamentally different? Probably not. All plans proved stable for 13-ft waves, while only Plans 3R and 3R1 proved stable for 16-ft waves. Past work of the present authors (Carver and Wright 1991) has shown that if a large number of repeat stability tests (10) are conducted for the same armor weight, structure geometry, and spectral wave conditions similar to those used in the present investigation, design wave heights can vary by up to 30 percent within a 90-percent confidence interval. Thus, the difference in results for the three plans may well be random.

Description of Plan 4

Plan 4 (Figure 7 and Photos 40-42) was the same as Plan 2, except an inner core was constructed from 25 percent sand and 75 percent quarry run stone. The purpose of the inner core was to reduce permeability and hence reduce long-period wave transmission. Plan 4 was checked for stability to make sure that the decreased permeability did not affect runup and overtopping enough to adversely affect stability.

Test Results of Plan 4

Plan 4 was tested with Hydrograph 3. No damage was observed during the first 10 steps. Step 11 displaced three landward armor stones. No additional damage was detected for the remaining 13-ft waves. As anticipated, step 15 produced a large damage increase. Damage was estimated to be 15 percent seaward and 40 percent landward at the conclusion of step 15 (Photos 43-45). Damage progressed during steps 16-18. At the conclusion of testing (Photos 46-48), the seaward armor from the area between mllw and the back of the crown had incurred 50 percent damage and landward damage had increased to about 75 percent, with some exposure of core and underlayer near the crest.

Test Results of Plan 4R

Plan 4R was identical to Plan 4. Gross statistics (T_p, H_{mo}, peak enhancement factor, etc.) of the spectra tested were the same as the initial stability test. However, as with previous repeat tests, spectral signals were randomly rephased for the repeat test. Similar to the initial stability test, little movement was detected during the first 10 steps. During steps 11 and 12 minor damage (2 percent) was incurred on the seaward face of the structure. The landward slope experienced significant damage (10 percent). Photos 49 and 50 show the structure at the conclusion of step 12. Damage continued to progress and testing was suspended after step 15. At the conclusion of testing (Photos 51 and 52), seaward armor from the area between mllw and the back of the crown had incurred about 30 percent damage and landward damage had increased to about 50 percent with exposure of core and underlayer near the crest. Initially, unexpected damage incurred for the 13-ft waves tested in steps 11 and 12 was vexing. However, inspection of the water

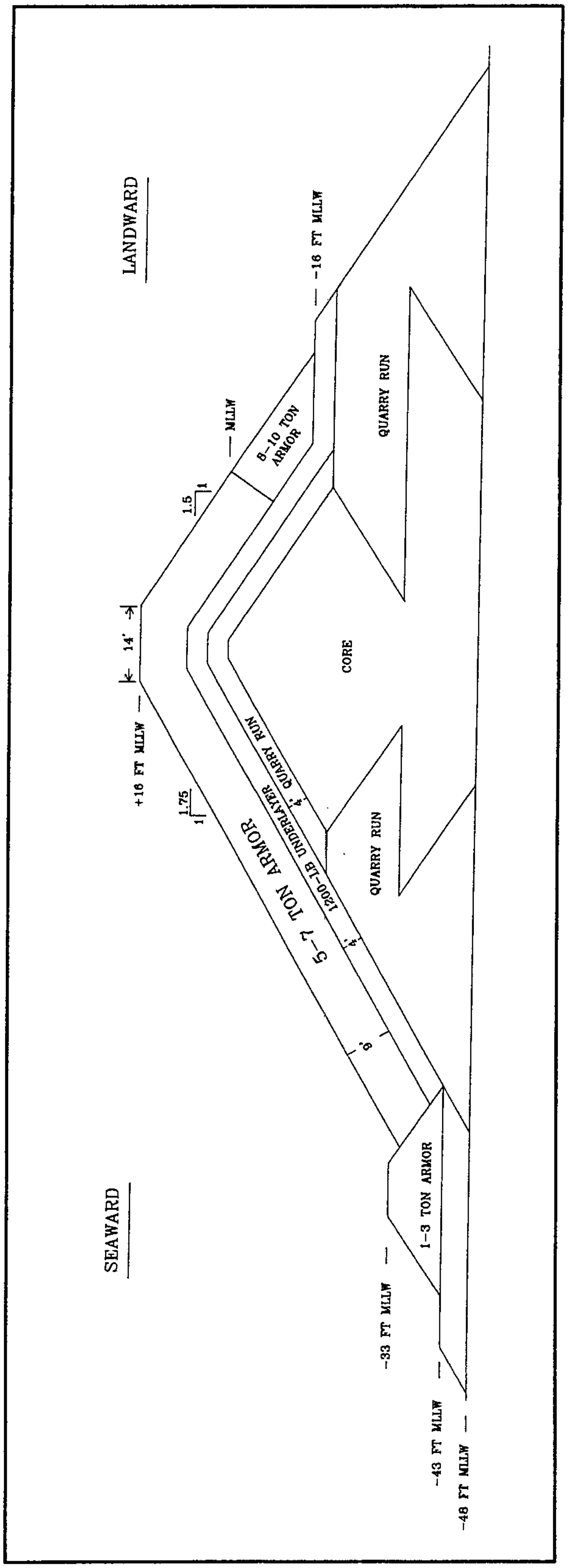

Figure 7. Characteristics of Plans 4, 4R, and 4R1

surface time-histories showed that the waves were very highly grouped for the new command signals, with four of the five largest waves occurring sequentially. Model observations and video reviews showed that unexpected damage occurred during attack of this wave group. Therefore, it was decided to conduct one more test in the Plan 4 series using both original signals and the grouped signals.

Test Results of Plan 4R1

Plan 4R1 was identical to Plans 4 and 4R. Plan 4R1 was tested with Hydrograph 3A, as shown in Table 6.

Table 6
Test Conditions for Hydrograph 3A

Step	swl, ft mllw	T_p, sec[1]	H_{mo}, ft[2]	Duration, hr
1	+1.0	12.0	6.0	3.0
2	+1.0	14.0	8.0	3.0
3	-1.0	16.0	9.0	3.0
4	+1.0	16.0	9.0	3.0
5	+3.0	16.0	9.0	3.0
6	+8.0	16.0	9.0	3.0
7	-1.0	18.0	11.0	3.0
8	+1.0	18.0	11.0	3.0
9	+3.0	18.0	11.0	3.0
10	+8.0	18.0	11.0	3.0
11	-1.0	18.0	13.0	3.0
12	+1.0	18.0	13.0	3.0
13	+3.0	18.0	13.0	3.0
14	+8.0	18.0	13.0	3.0
15	-1.0	18.0	13.0[3]	3.0
16	+1.0	18.0	13.0[3]	3.0
17	+3.0	18.0	13.0[3]	3.0
18	+8.0	18.0	13.0[3]	3.0
19	-1.0	18.0	16.0	3.0
20	+1.0	18.0	16.0	3.0
21	+3.0	18.0	16.0	3.0
22	+8.0	18.0	16.0	3.0

[1] T_p = Wave period of peak energy density of spectrum, sec
[2] H_{mo} = Zero-moment wave height, ft
[3] Highly grouped.

Hydrograph 3A was the same as Hydrograph 3, except the 13-ft grouped waves were inserted (steps 15-18). No damage was observed during the first 10 steps. The 13-ft ungrouped waves tested in steps 11-14 caused minor (1.5 percent) landward damage. Photos 53 and 54 show the breakwater after step 14. As anticipated, steps 15-18 produced a moderate damage increase. Damage was estimated to be 2 percent seaward and 8 percent landward at the conclusion of step 18 (Photos 55 and 56). As anticipated, damage progressed during steps 18-22. At the conclusion of testing (Photos 57-59), the seaward armor from the area between mllw and the back of the crown had incurred 5 percent damage and landward damage had increased to about 25 percent.

4 Wave Transmission Tests and Results

Large-Scale Tests

Limited wave transmission tests were conducted in the 1:30 scale model. Results of these tests were later used to help develop distorted scale breakwater sections (1:100 vertically and 1:400 horizontally) for eventual use in the three-dimensional harbor model. All transmission tests were conducted with uniform spectra having period limits of 20 and 120 sec. Tests were conducted with amplitudes of 1 to 4 ft, which corresponded to electronic gains in the 10- to 40-percent range.

Transmission tests were conducted on Plans 2 and 4. Plans 2 and 4 were similar, except Plan 4 utilized an inner core which was constructed from 25 percent sand and 75 percent quarry run stone. The purpose of the inner core was to reduce permeability and hence reduce long-period wave transmission. Signals 1, 2, and 3 had approximately the same total energy; however, energy was redistributed in Signals 2 and 3 in an effort to achieve a more nearly uniform transmitted spectra. Test results were as shown in Table 7. As shown in Table 7 and in Figure 8, Plan 4 consistently produced the best transmission performance.

Distorted-Scale Tests

Distorted-scale transmission tests were conducted in the previously described 3-ft flume. These tests were conducted at a scale of 1:100 vertical and 1:400 horizontal. Distorted model breakwater sections constructed of No. 3-No. 4 stone, No. 4-No. 6 stone, and small glass beads with an average diameter 0.10 in. were tested with the results as shown in Table 8.

The information in Table 8 and results for the optimum signal from the large-scale transmission tests are presented in Figure 9. These data show that the No. 4-No. 6 stone breakwater sections closely match the large-scale transmission test results; therefore, the No. 4-No. 6 stone is recommended for use in the three-dimensional distorted model.

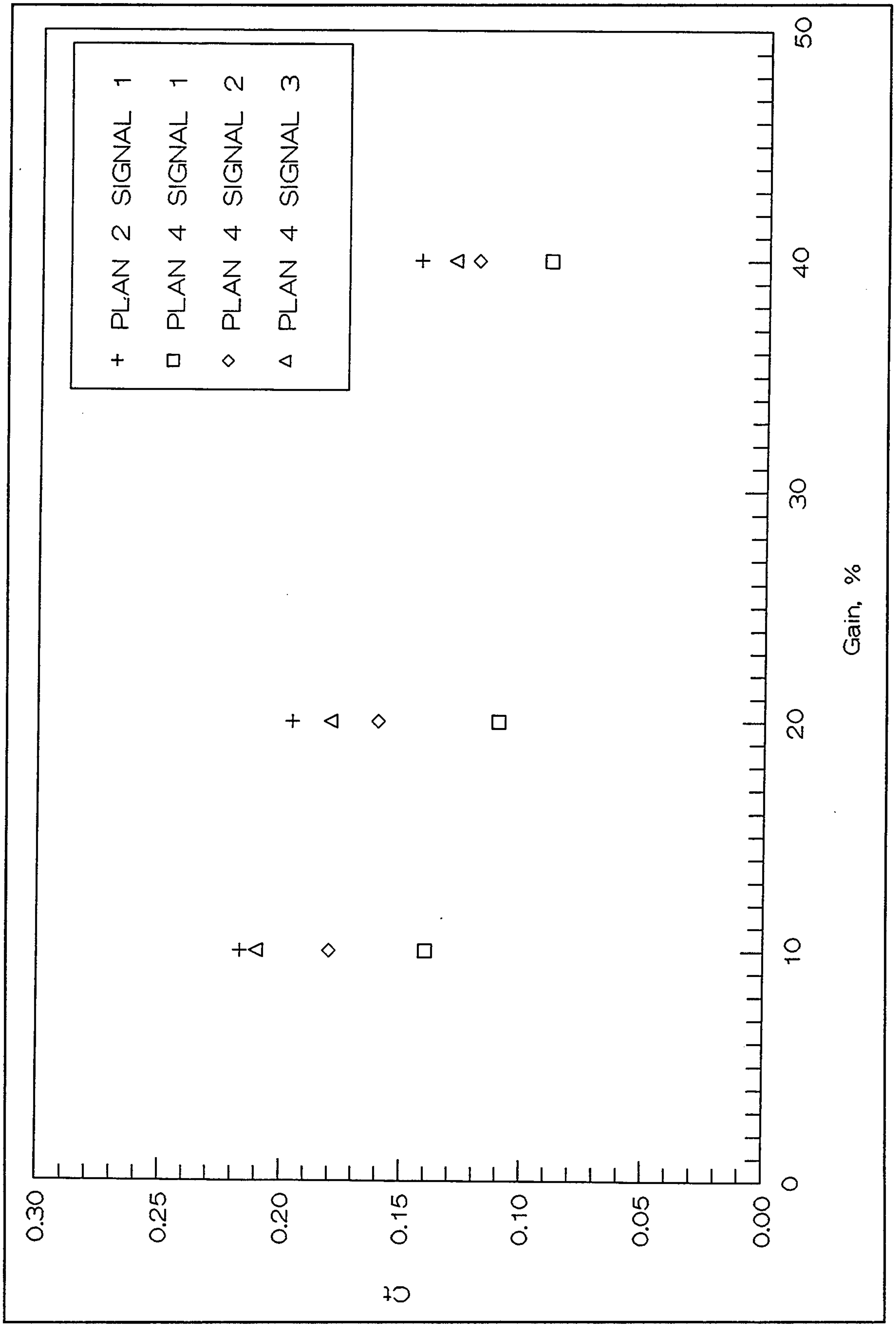

Figure 8. Transmission coefficient (Ct) versus gain

Table 7
Large-Scale Transmission Test Results

Plan	Signal	Gain, %	Transmission Coefficient
2	1	10	0.22
2	1	20	0.20
2	1	40	0.14
4	1	10	0.14
4	1	20	0.11
4	1	40	0.09
4	2	10	0.18
4	2	20	0.16
4	2	40	0.12
4	3	10	0.21
4	3	20	0.18
4	3	40	0.13

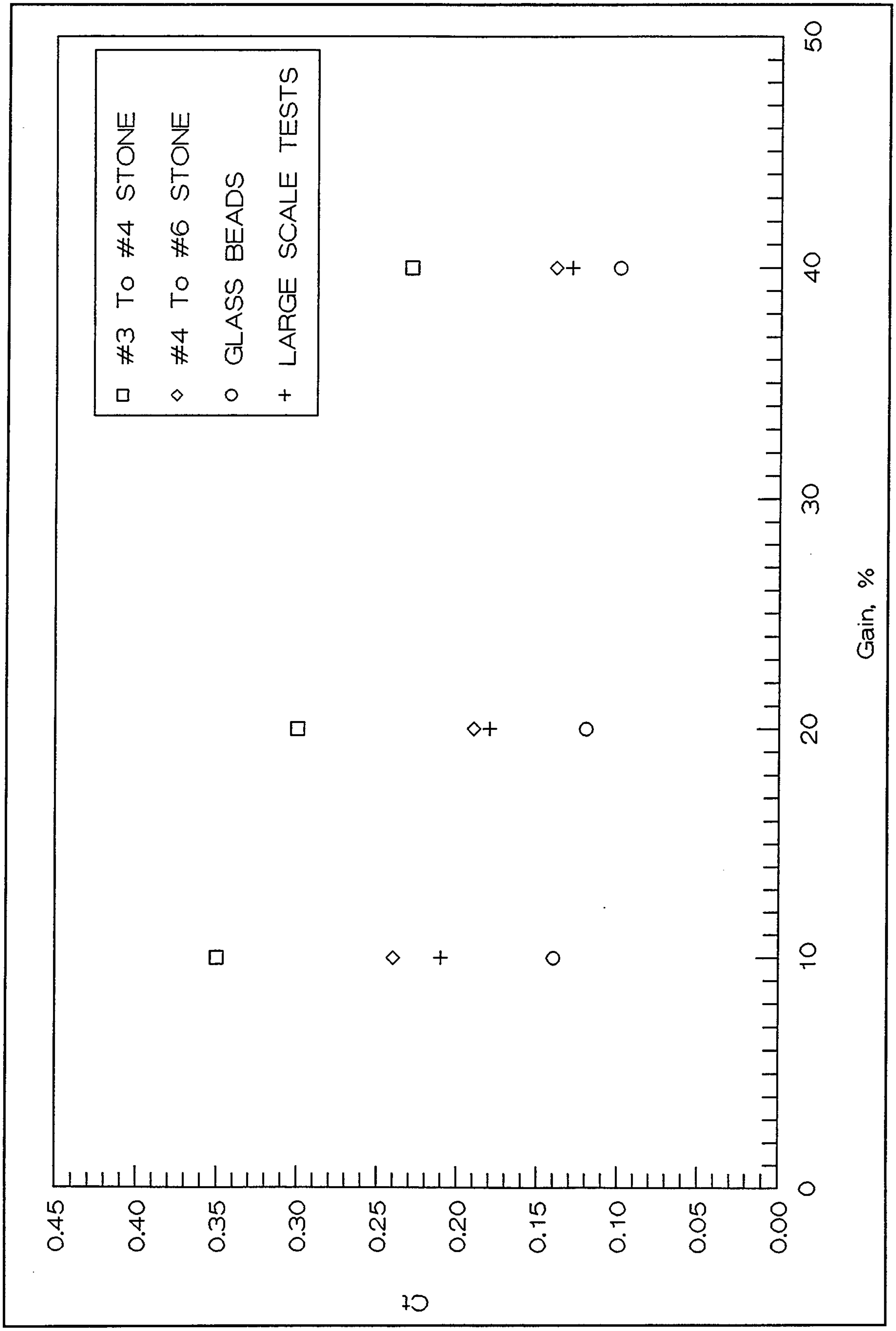

Figure 9. Transmission coefficient (Ct) versus gain (distorted scale model)

5 Conclusions

Based on assumptions, tests, and results reported herein, it is concluded that:

a. Testing of Plan 1 with Hydrograph 1 showed the structure to be a stable design for maximum wave heights of 13 and 16 ft at swl's of 0 and +8 ft mllw, respectively.

b. Plan 1R experienced major landward damage for 13-ft waves at an swl of 0 ft mllw when tested with Hydrograph 2.

c. Testing of Plans 2 and 3 with Hydrograph 3 showed the structures to be stable for a maximum wave height of 13 ft at swl's of -1, +1, +3, and +8 ft mllw.

d. Testing of Plan 3R with Hydrograph 3 showed the structure to be stable for a maximum wave height of 16 ft at swl's of -1, +1, +3, and +8 ft mllw.

e. Testing of Plan 3R1 with Hydrograph 4 showed the structure to be stable for a maximum wave height of 16 ft at swl's of -1, +1, +3, and +8 ft mllw.

f. Testing of Plan 4 with Hydrograph 3 showed the structure to be stable for a maximum wave height of 13 ft at swl's of -1, +1, +3, and +8 ft mllw.

g. Plan 4R test results show that highly grouped 13-ft waves may produce significant damage.

h. Test results for Plan 4R1 verify both Plan 4 and 4R results, i.e., the 5-to 7- ton stone showed significant damage at the 13-ft wave height level only when subjected to the highly grouped waves.

i. Stability of armor stone above the swl appears to be highly dependent on placement technique; therefore, it is very important to replicate the special placement used in the model for all prototype construction.

j. Transmission test results for Plans 2 and 4 show that the inner core used on Plan 4 (constructed from 25 percent sand and 75 percent quarry run stone) reduced permeability and hence reduced long-period wave transmission.

k. Distorted-scale transmission test results show that the No.4-No.6 stone breakwater sections closely match the large-scale transmission test results; therefore, the No.4-No.6 stone is recommended for use in the three-dimensional distorted model.

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Carver, R. D., and Wright, B. J. (1991). "Investigation of random variations in stability response of stone-armored, rubble-mound breakwaters," Technical Report CERC-91-17, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

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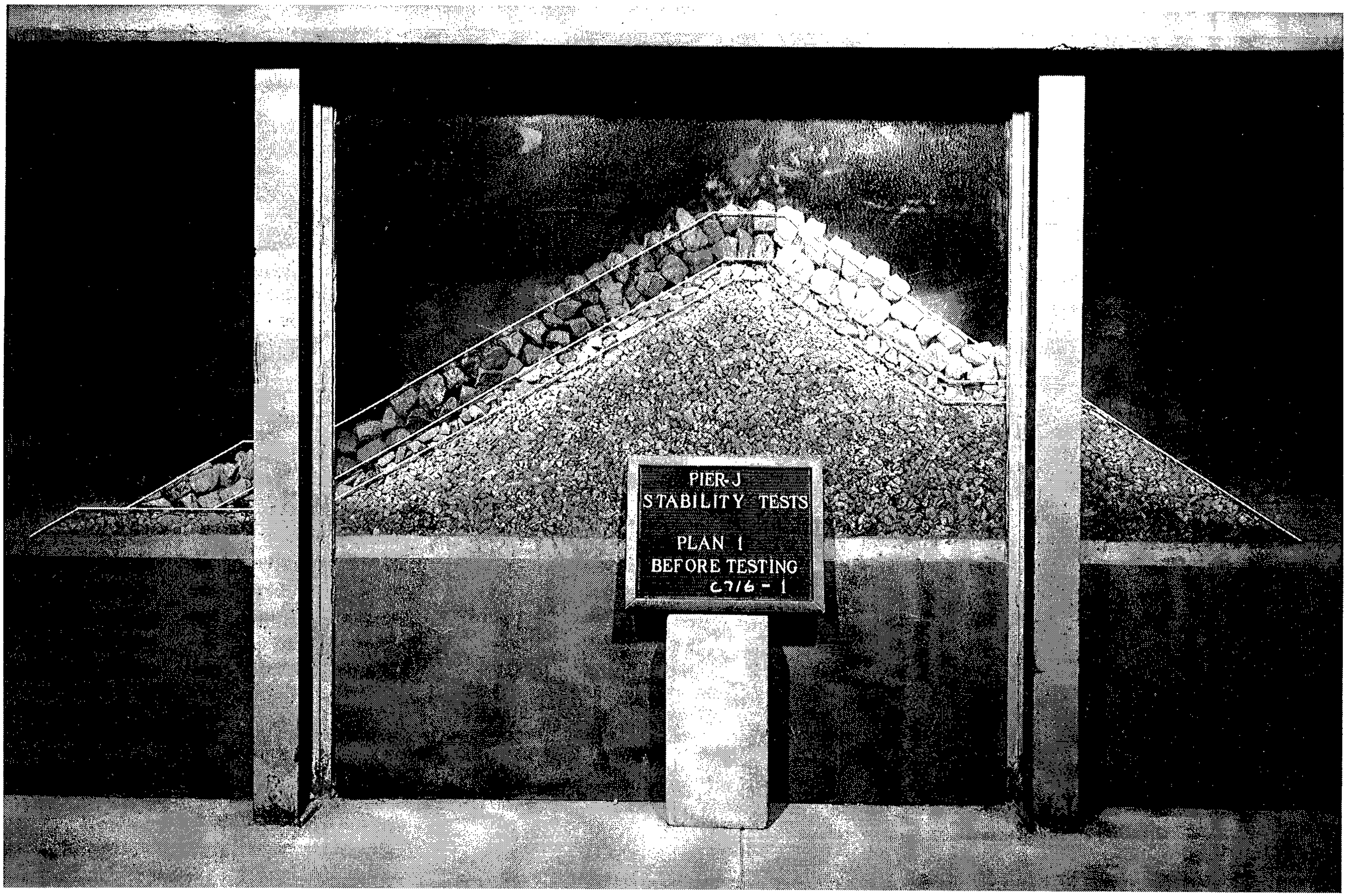


Photo 1. End view of Plan 1 before wave attack

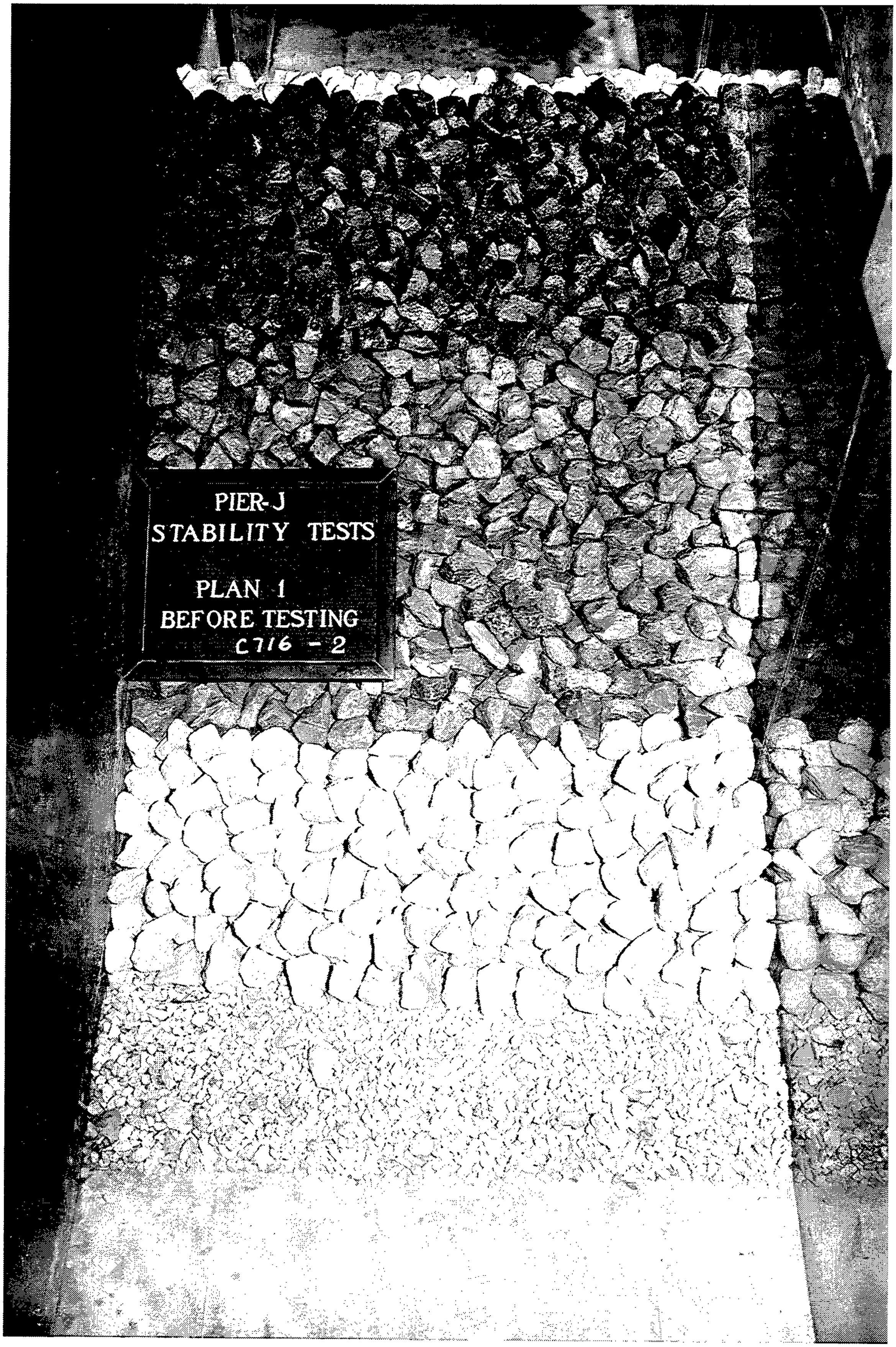


Photo 2. Sea-side view of Plan 1 before wave attack

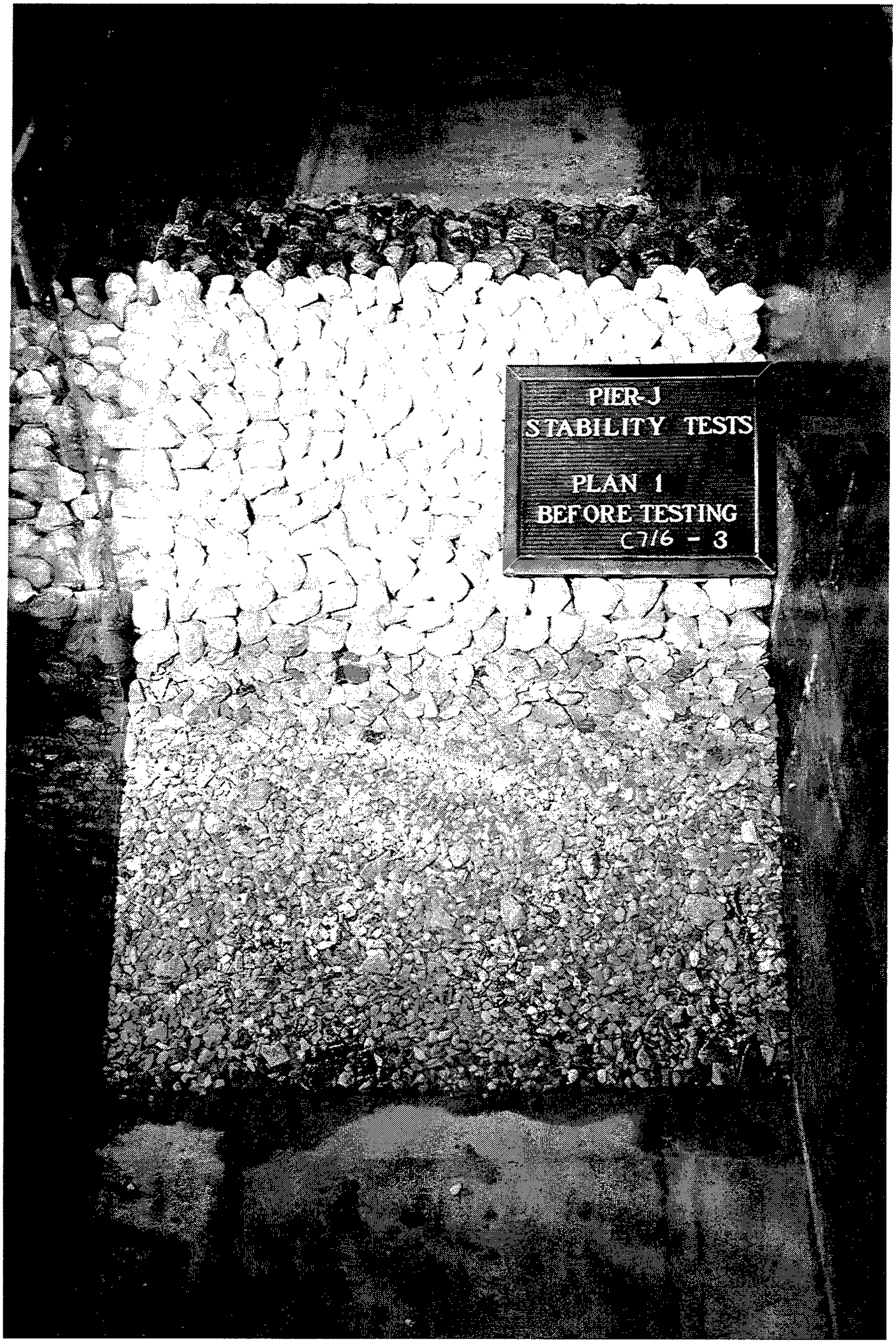


Photo 3. Harbor-side view of Plan 1 before wave attack

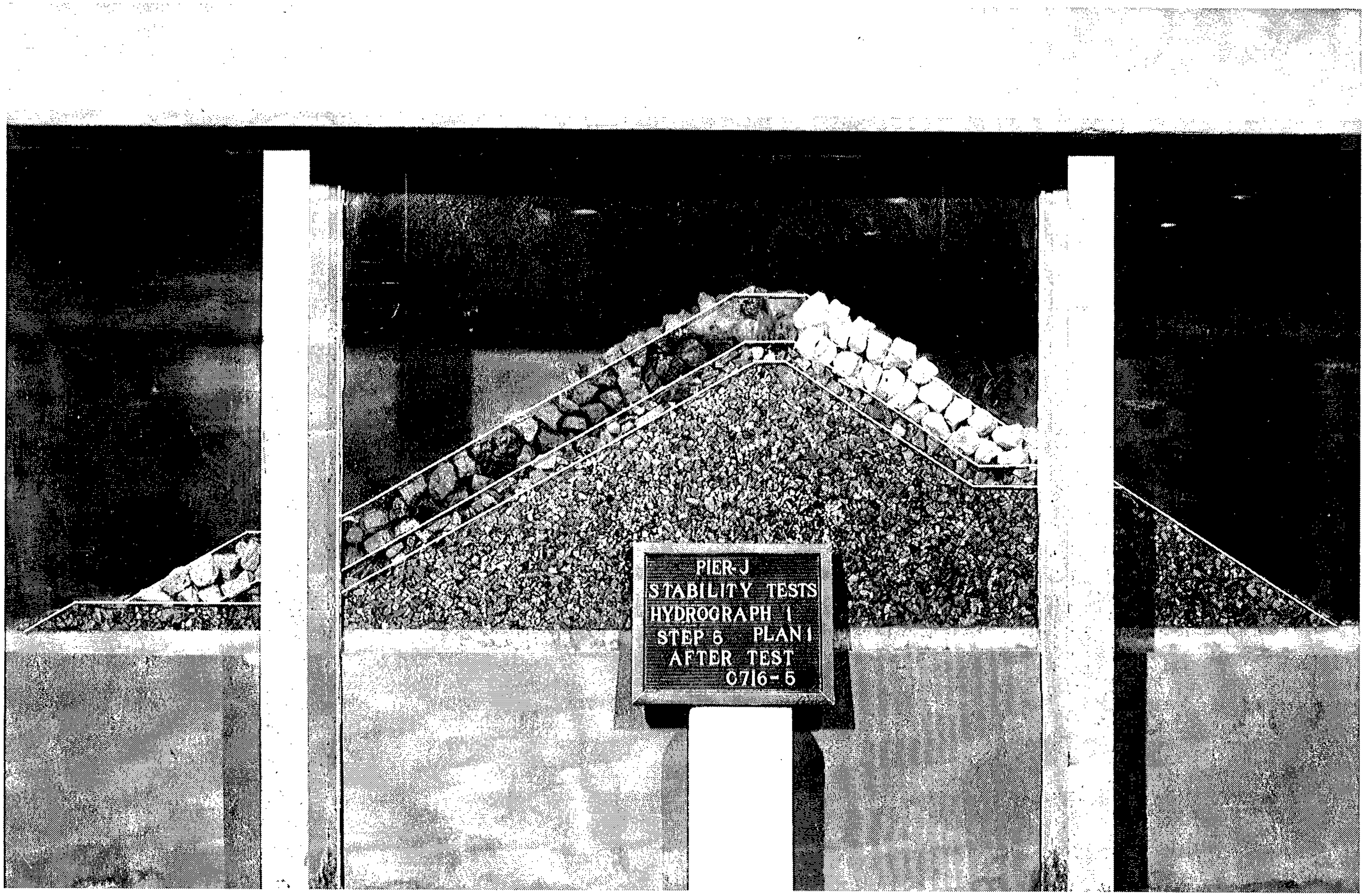


Photo 4. End view of Plan 1 after testing step 5 of Hydrograph 1

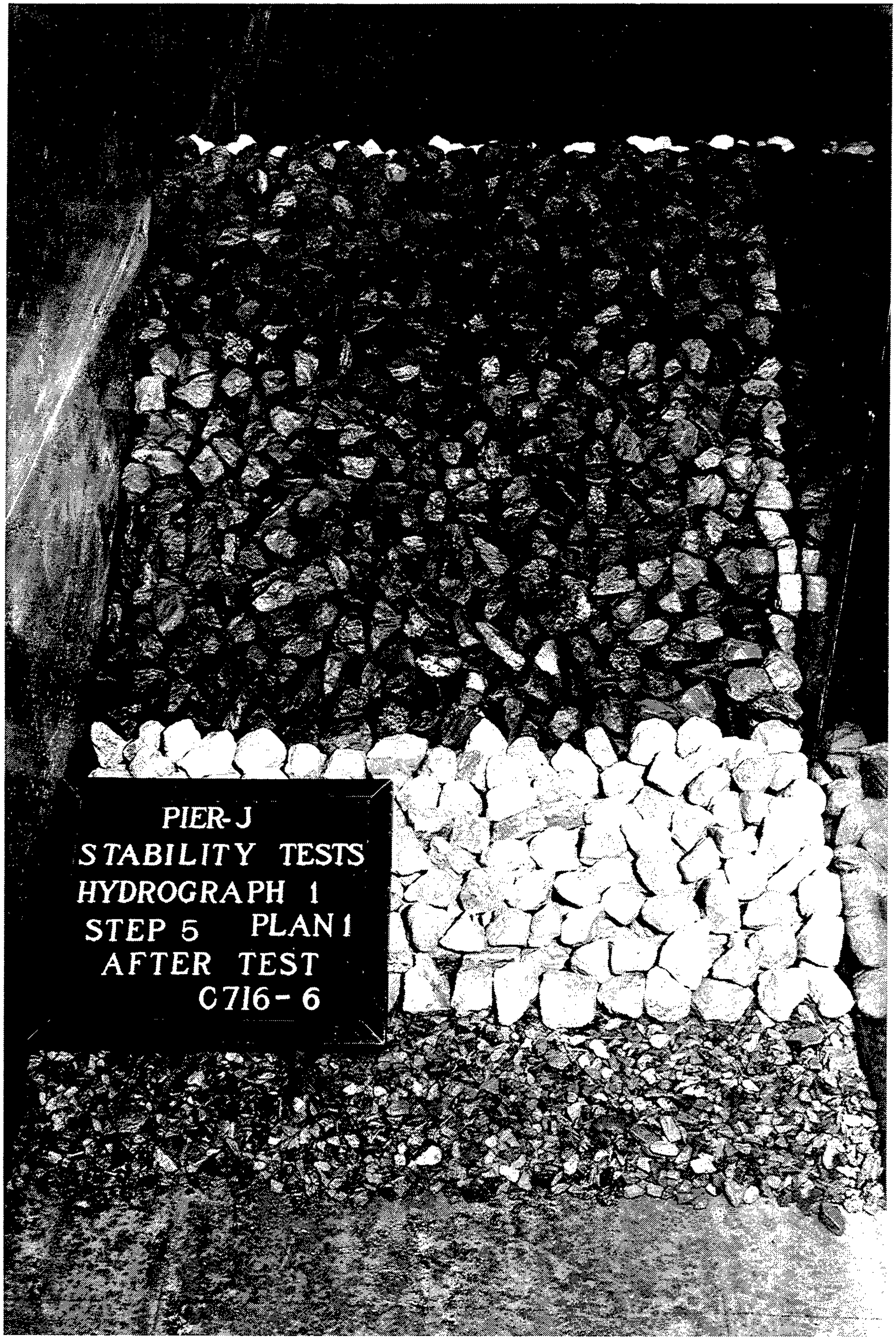


Photo 5. Sea-side view of Plan 1 after testing step 5 of Hydrograph 1

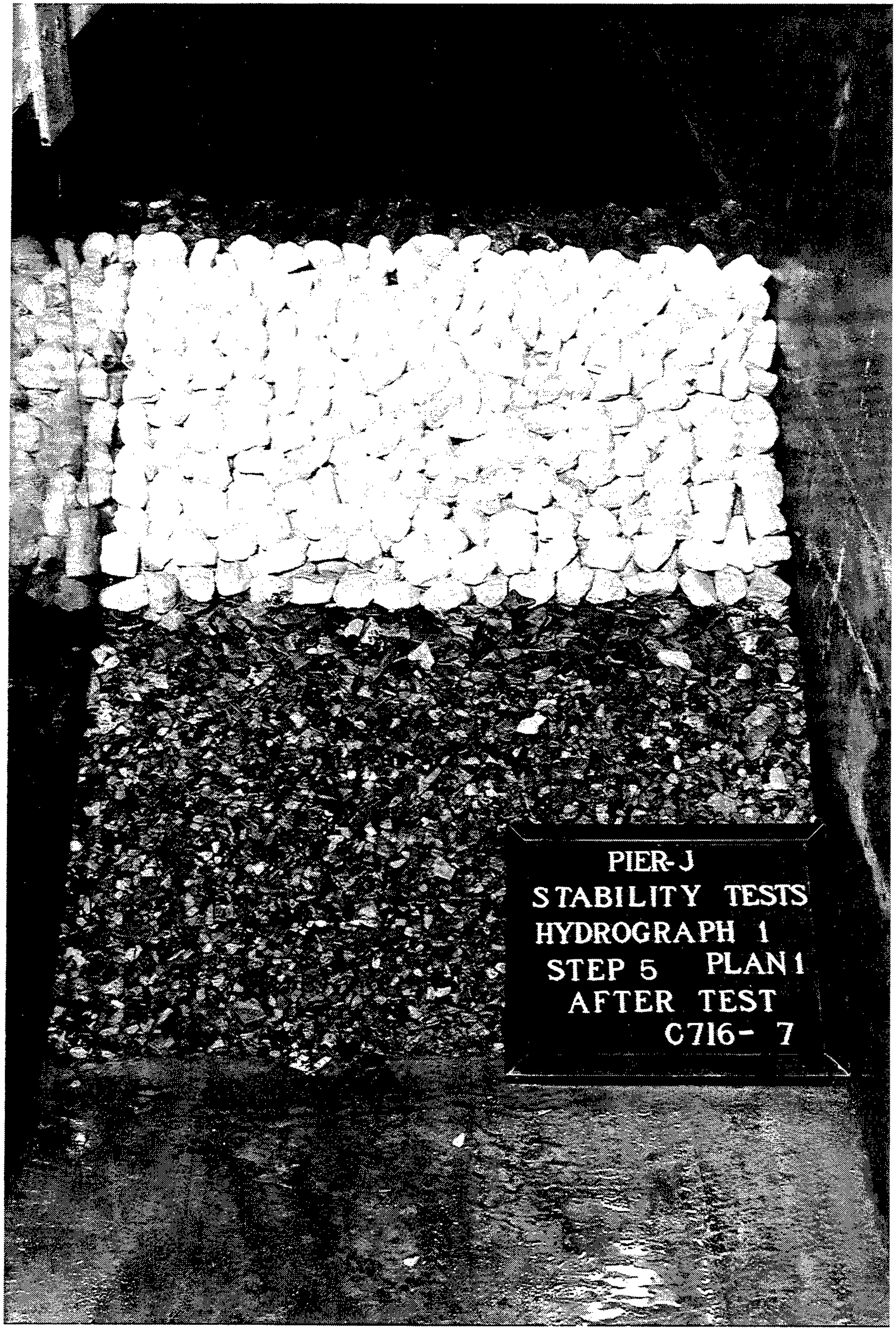


Photo 6. Harbor-side view of Plan 1 after testing step 5 of Hydrograph 1

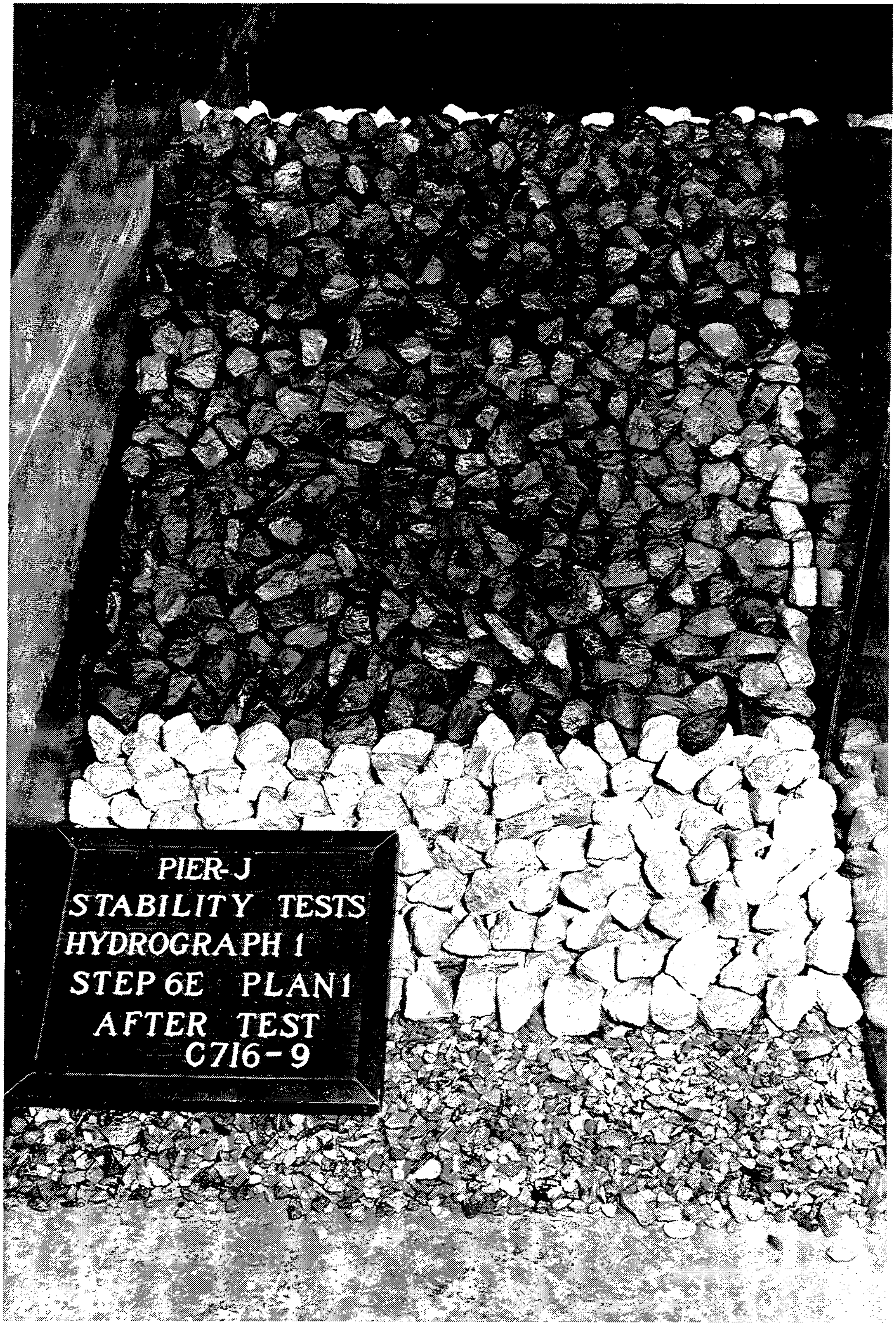


Photo 7. Sea-side view of Plan 1 after testing step 6 of Hydrograph 1

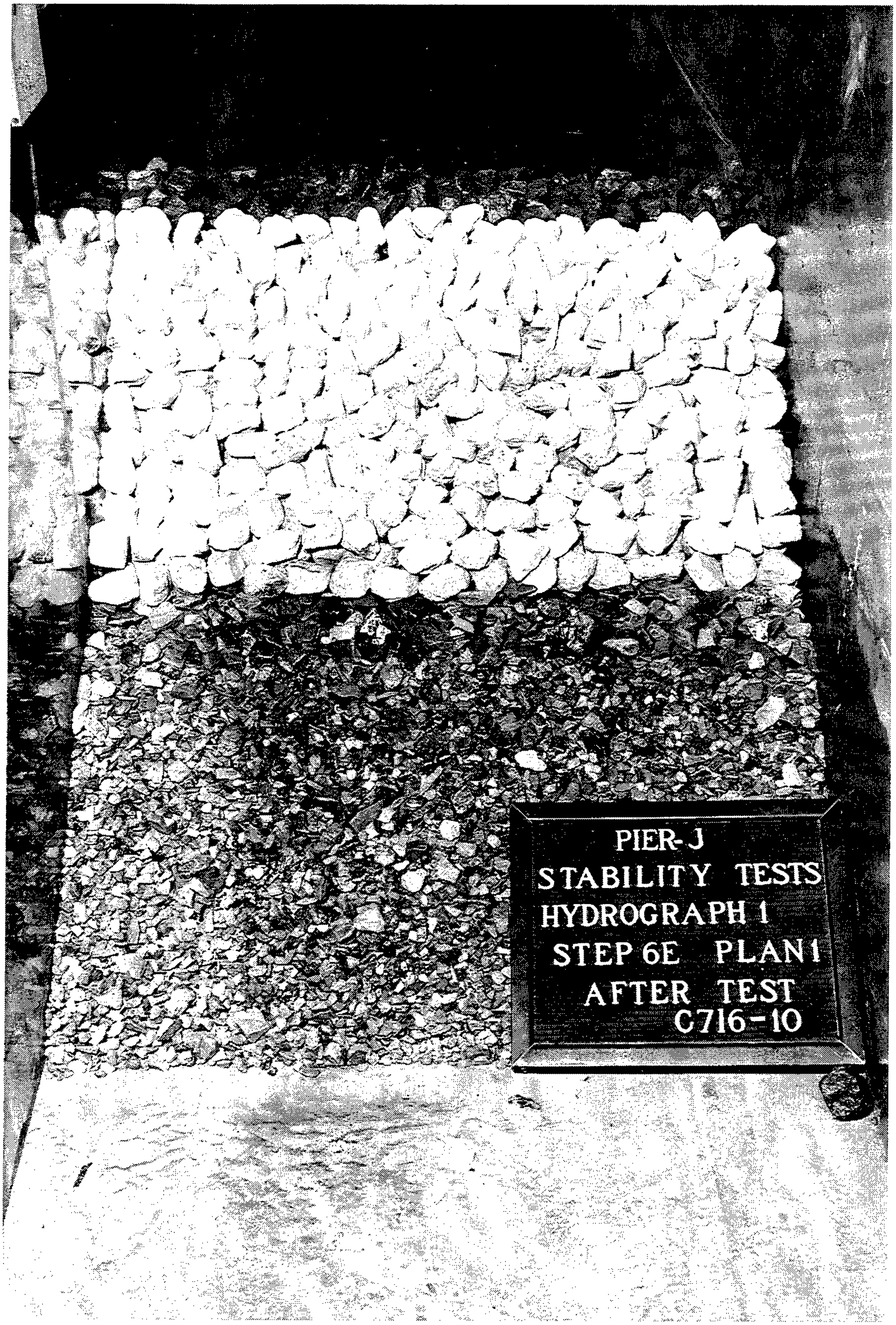


Photo 8. Harbor-side view of Plan 1 after testing step 6 of Hydrograph 1

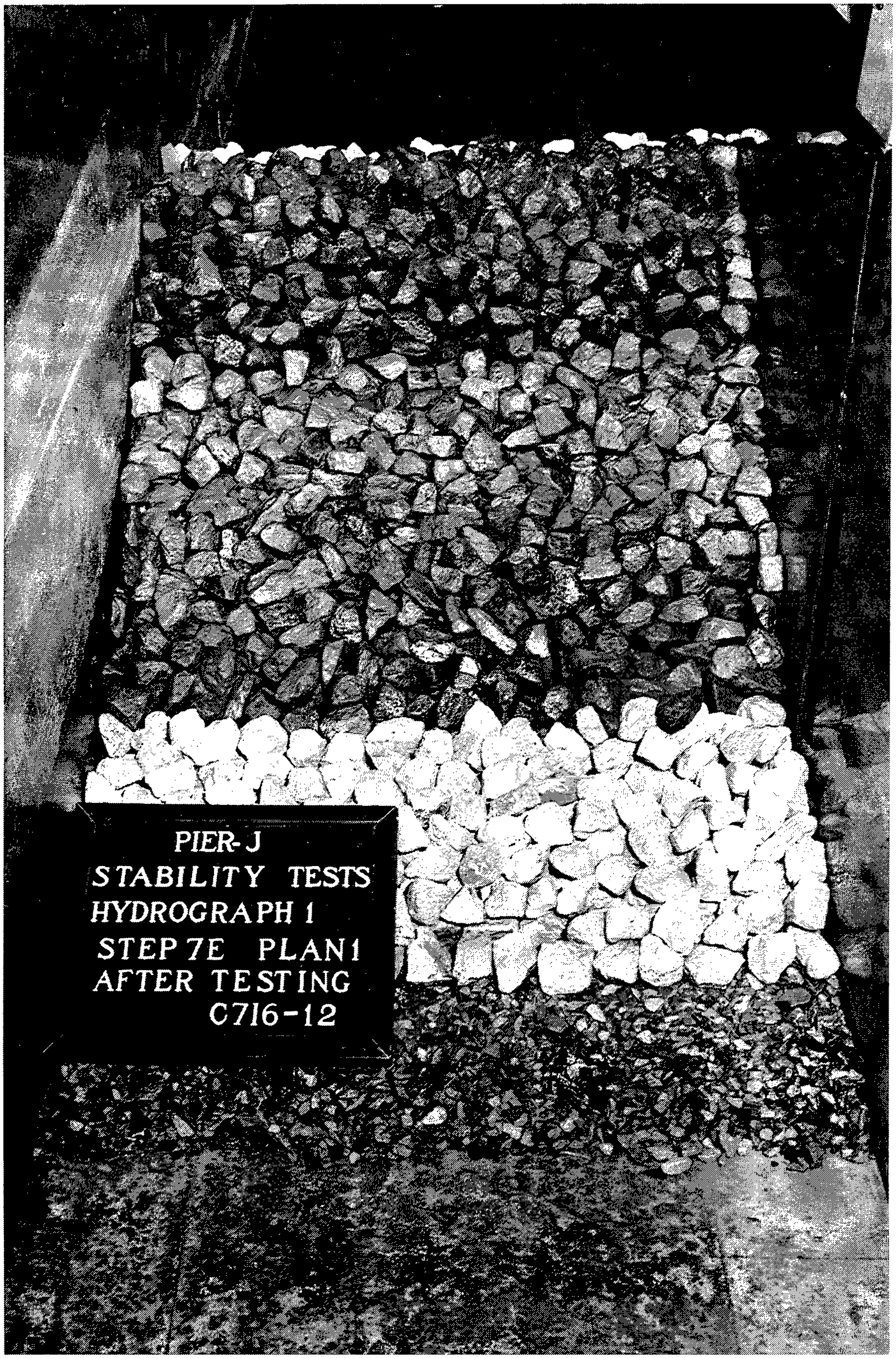


Photo 9. Sea-side view of Plan 1 after testing step 7 of Hydrograph 1

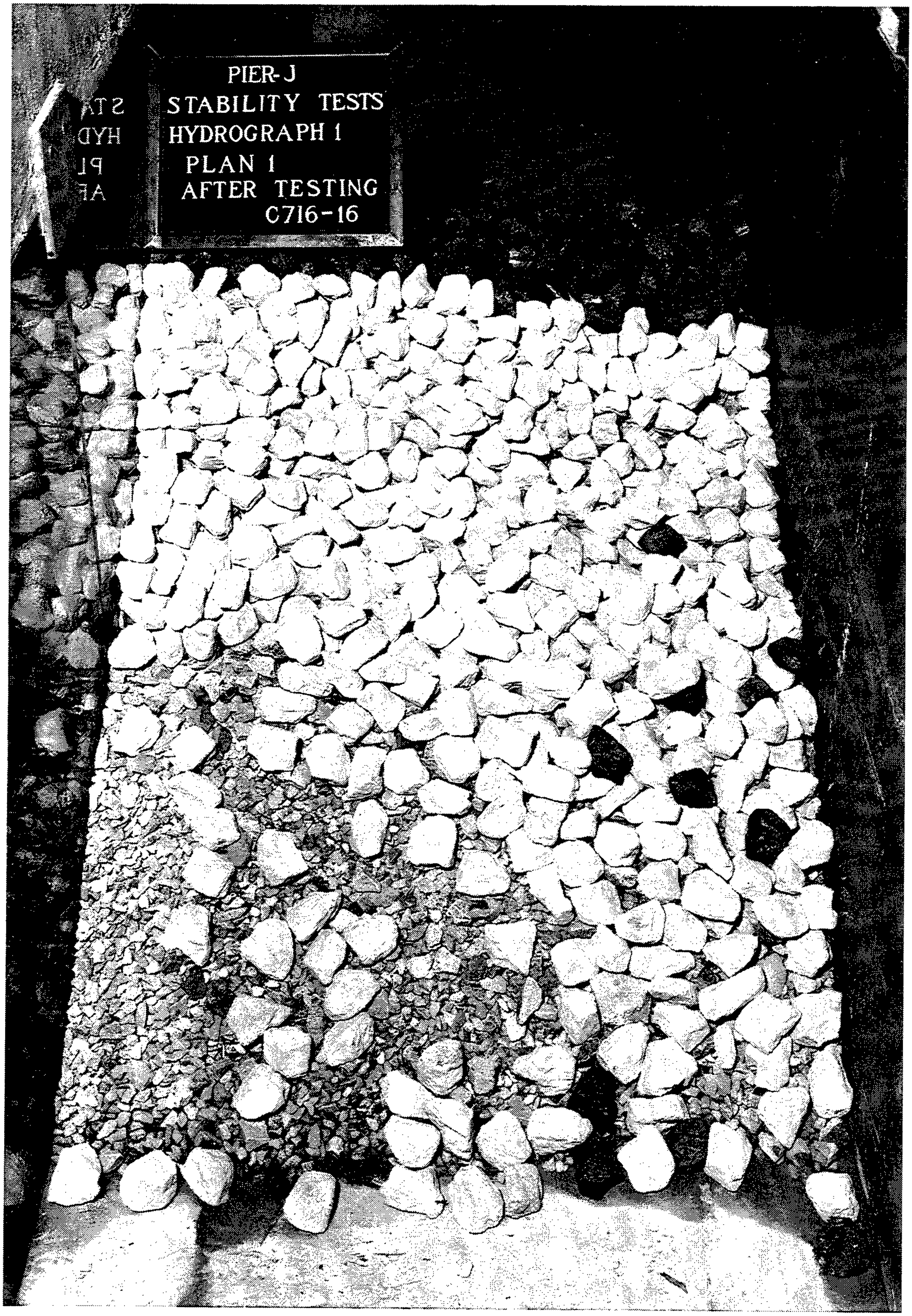


Photo 10. Harbor-side view of Plan 1 at the conclusion of testing

Photo 11. Sea-side view of Plan 1 at the conclusion of testing

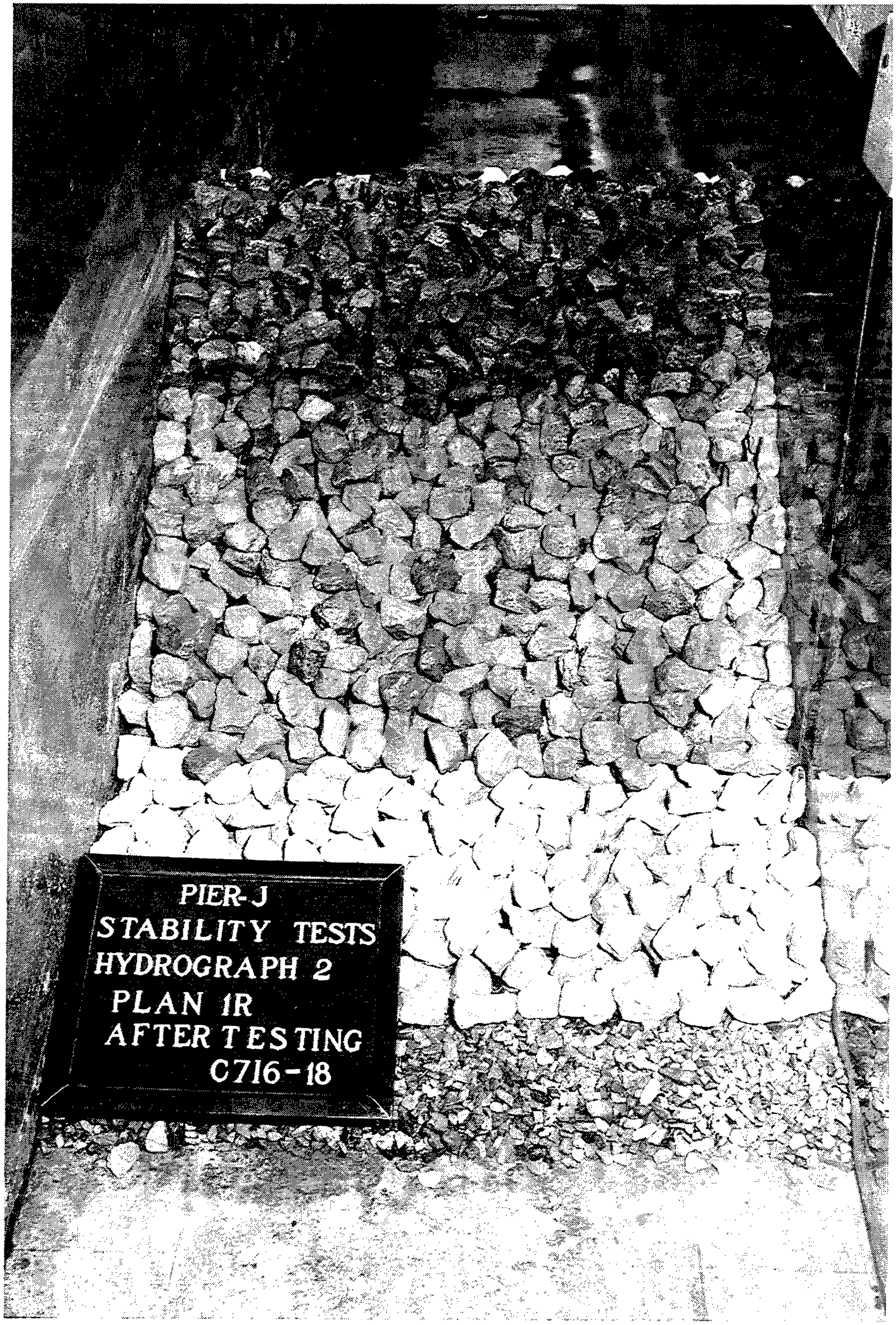


Photo 12. Sea-side view of Plan 1R after testing step 5 of Hydrograph 2

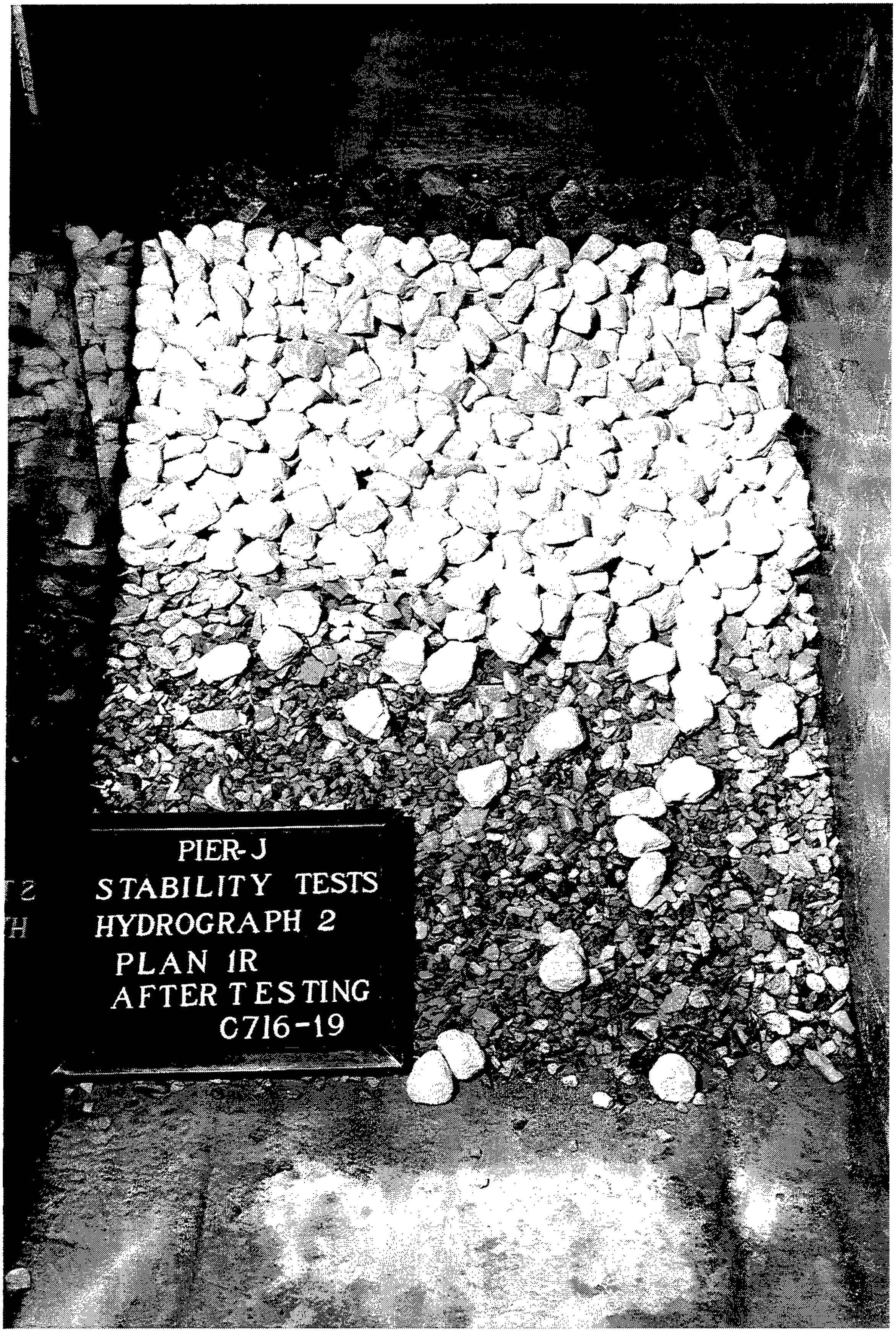


Photo 13. Harbor-side view of Plan 1R after testing step 5 of Hydrograph 2

Photo 14. End view of Plan 1R at the conclusion of testing

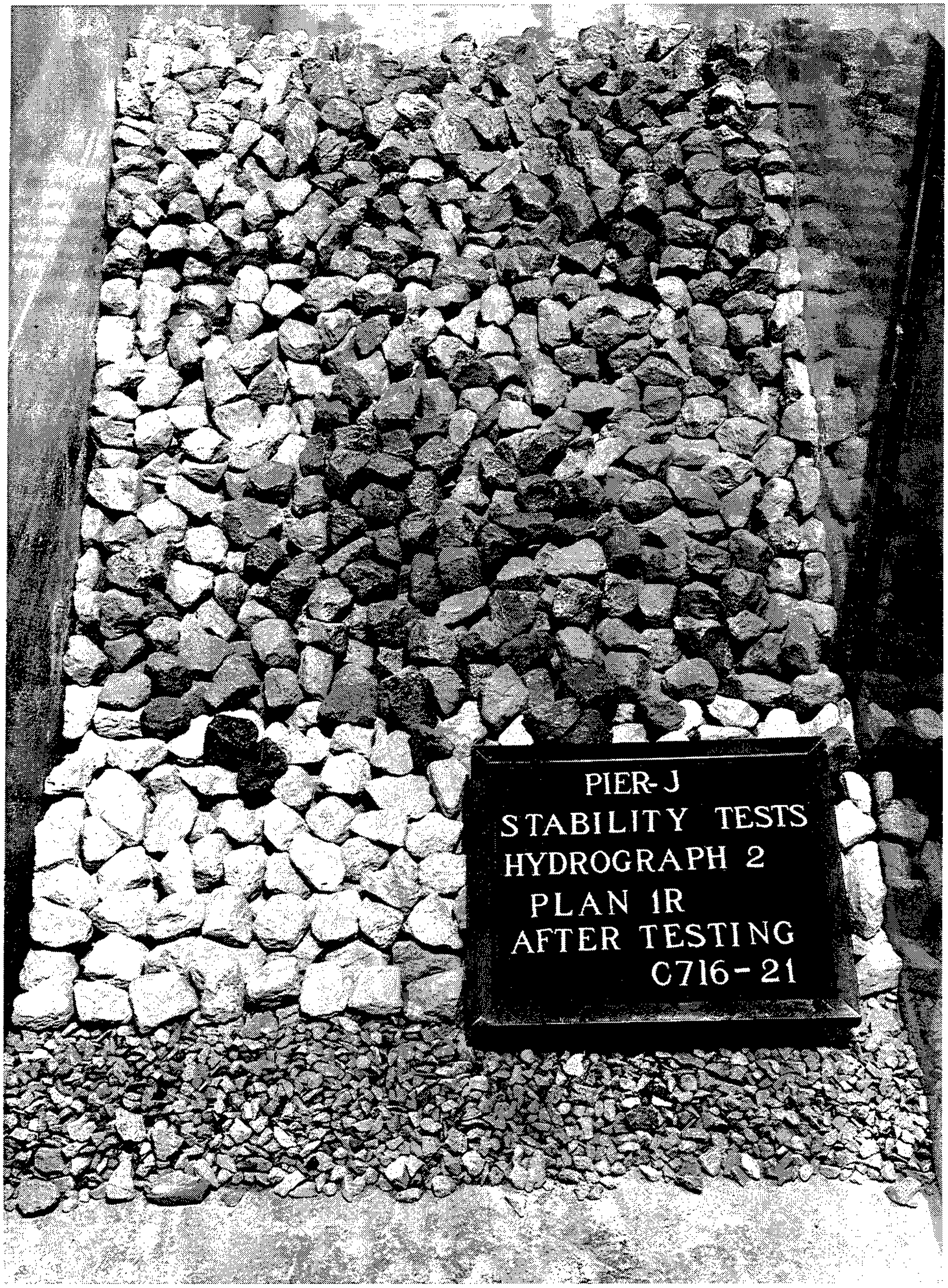


Photo 15. Sea-side view of Plan 1R at the conclusion of testing

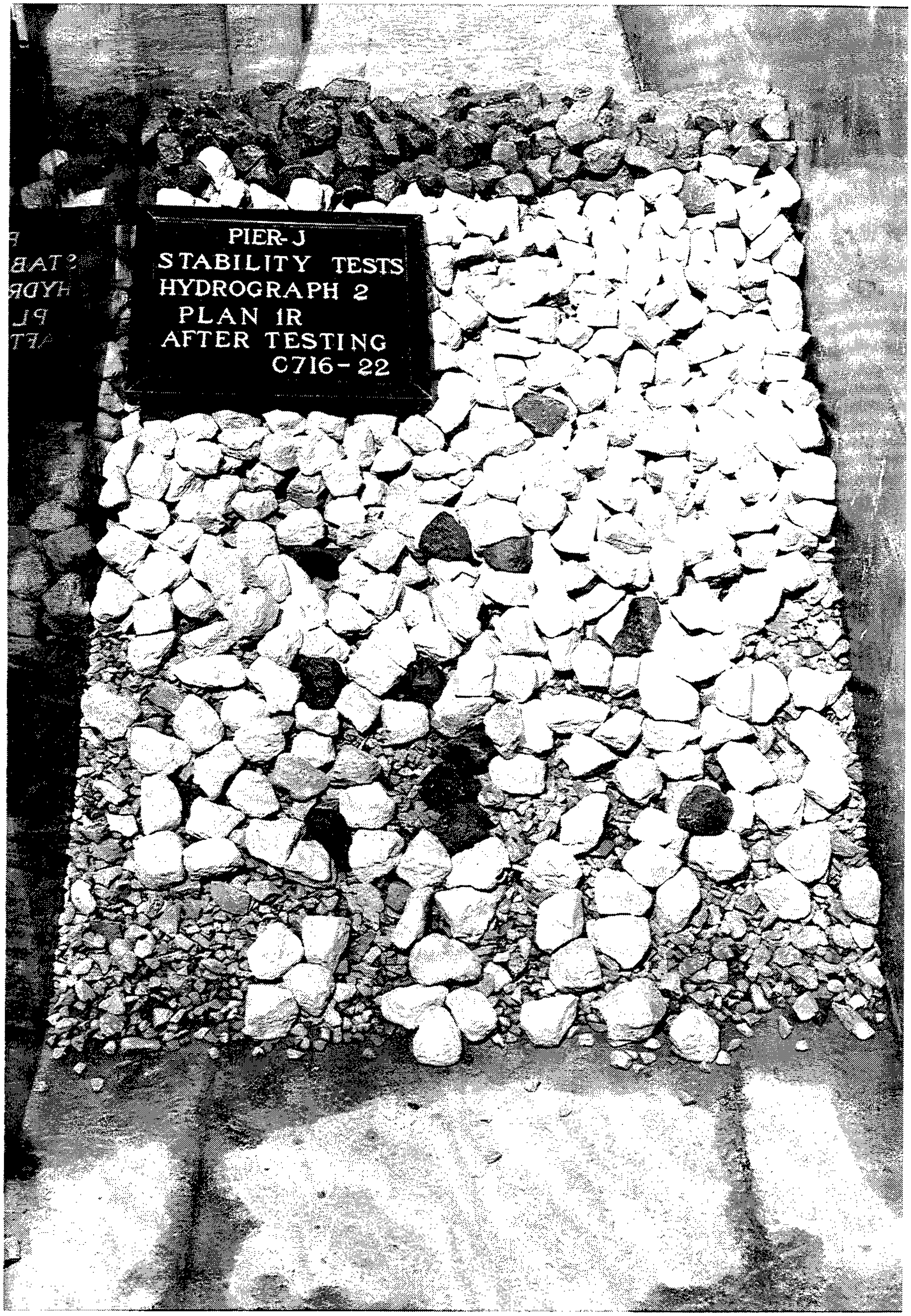


Photo 16. Harbor-view of Plan 1R at the conclusion of testing

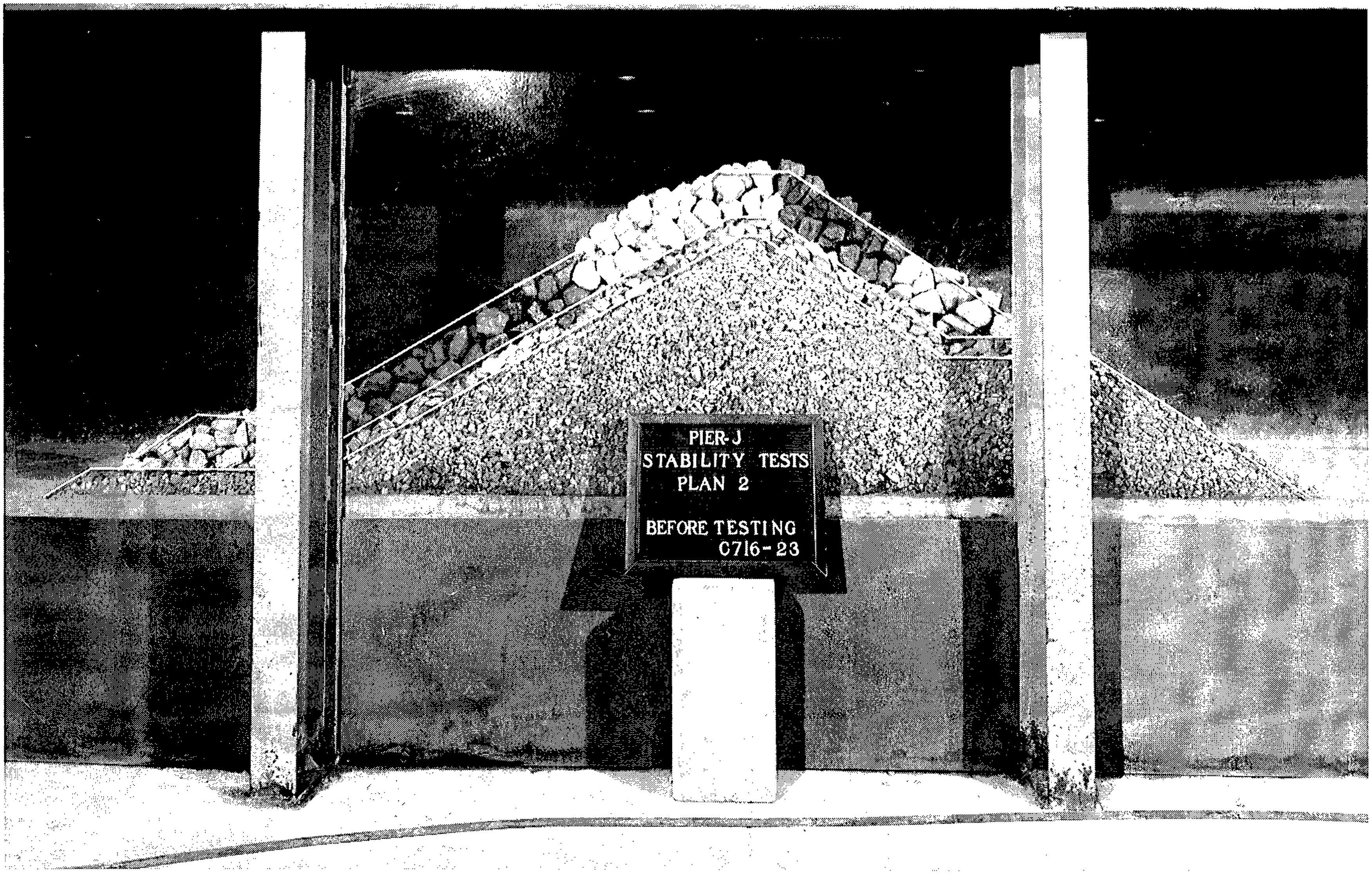


Photo 17. End view of Plan 2 before wave attack

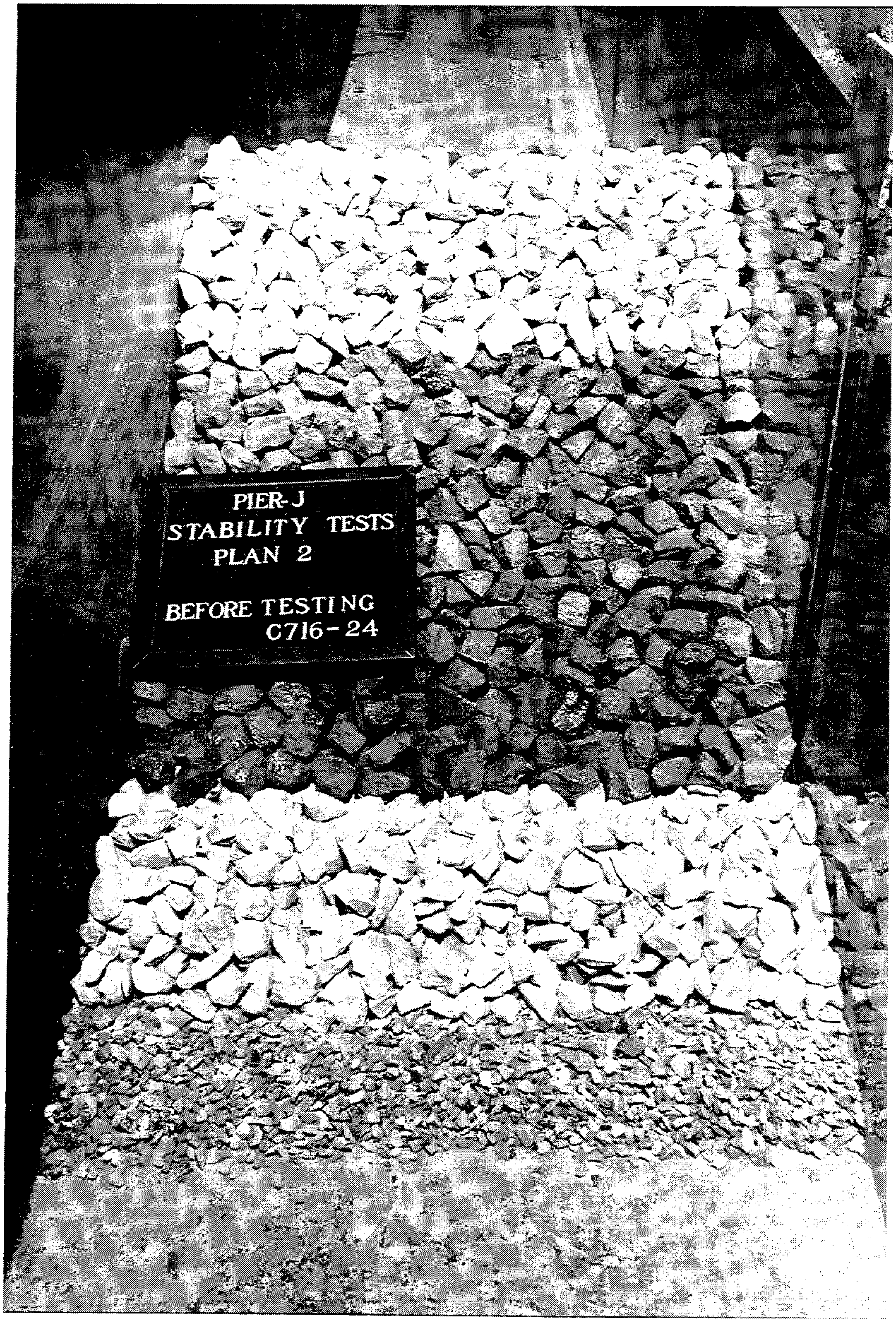


Photo 18. Sea-side view of Plan 2 before wave attack

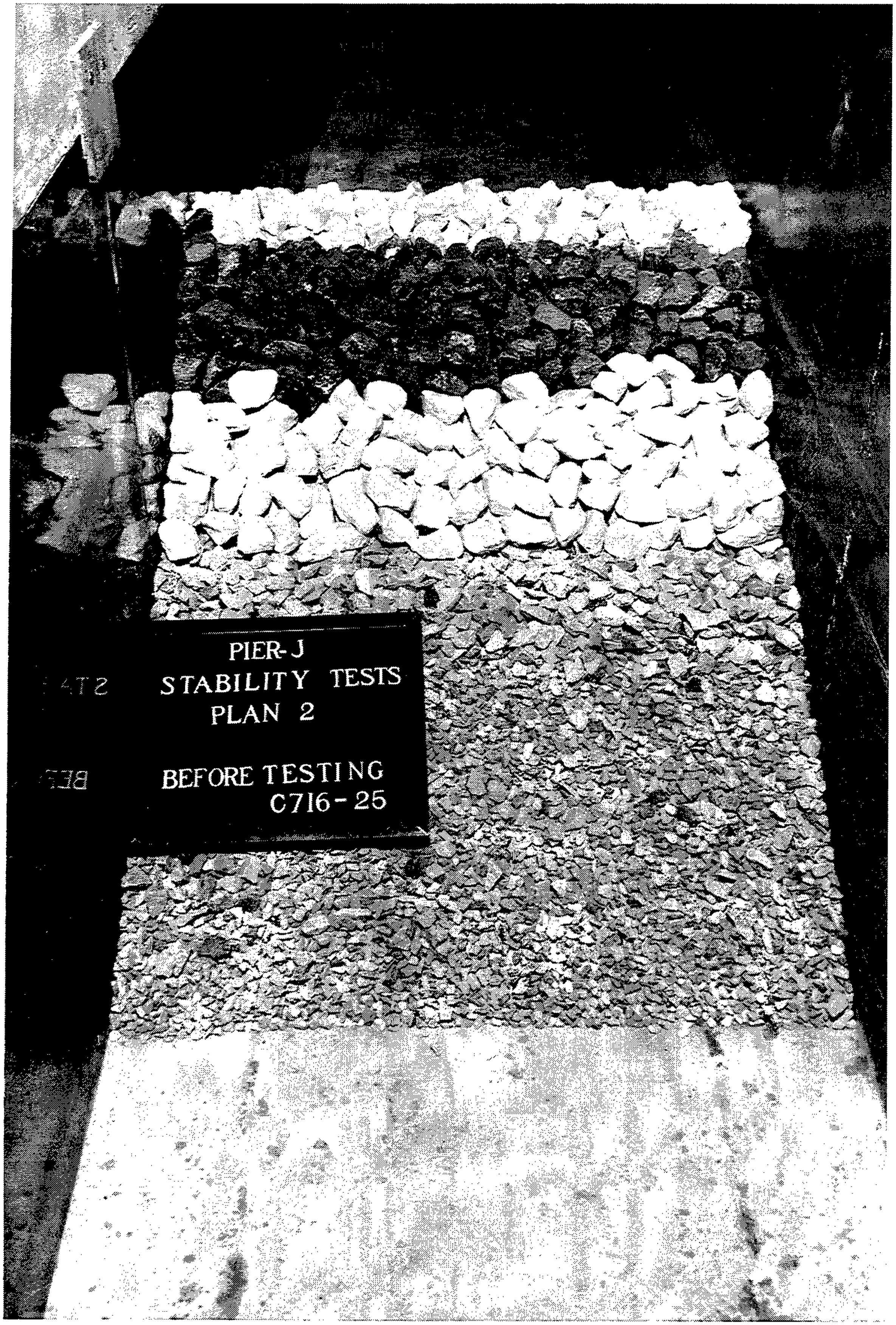


Photo 19. Harbor-side view of Plan 2 before wave attack

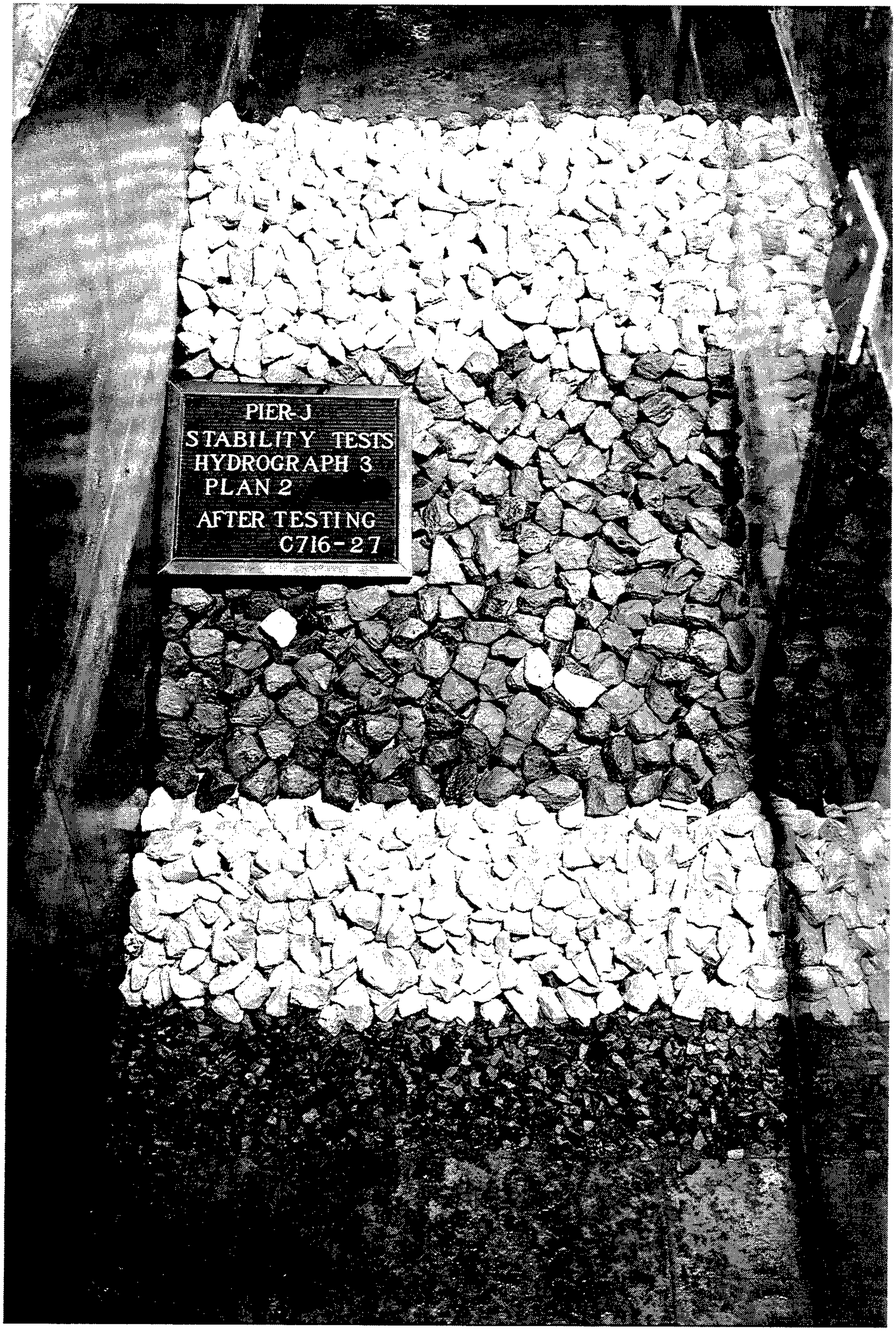


Photo 20. Sea-side view of Plan 2 after testing step 4 of Hydrograph 3

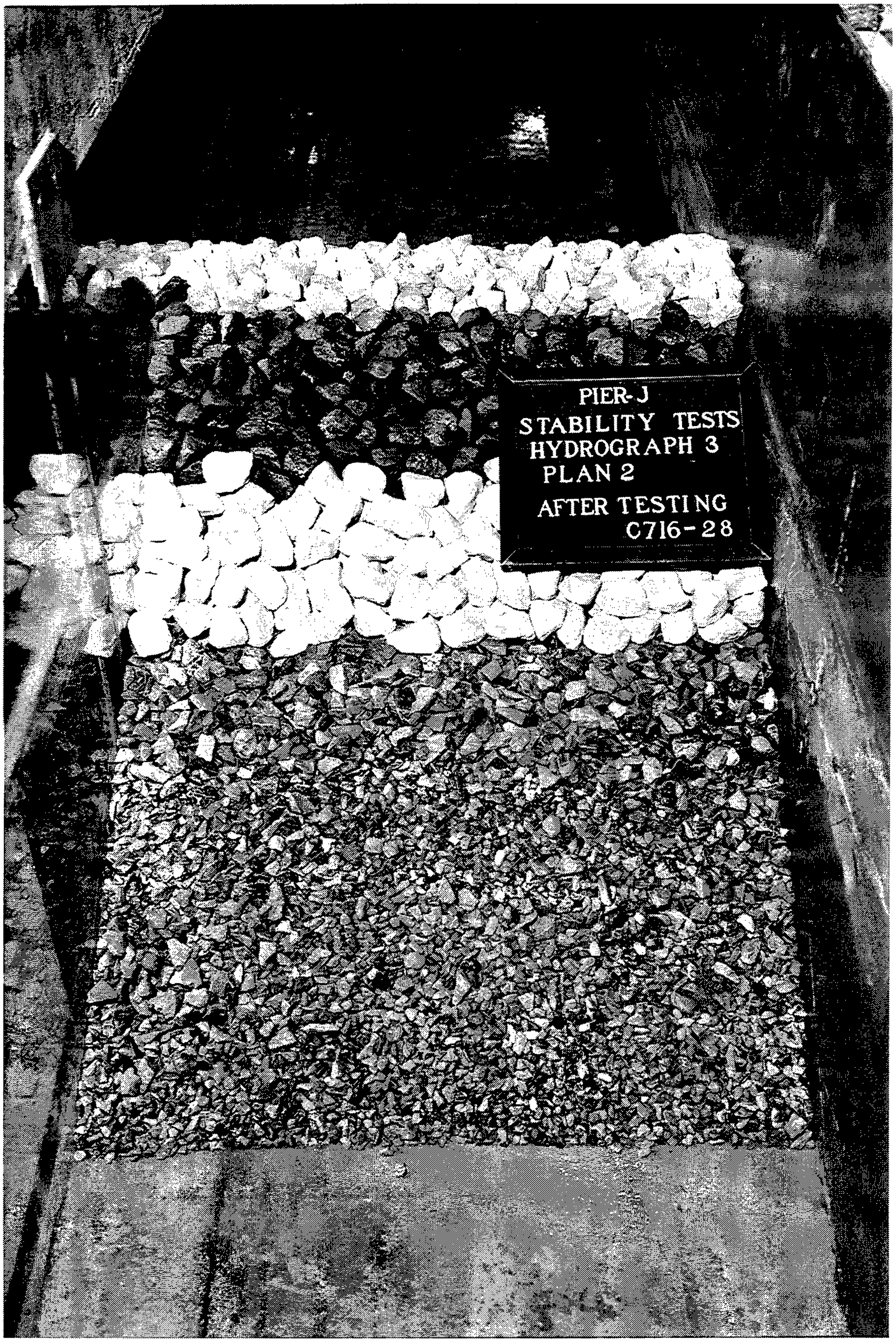


Photo 21. Harbor-side view of Plan 2 after testing step 4 of Hydrograph 3

Photo 22. End view of Plan 2 at the conclusion of testing

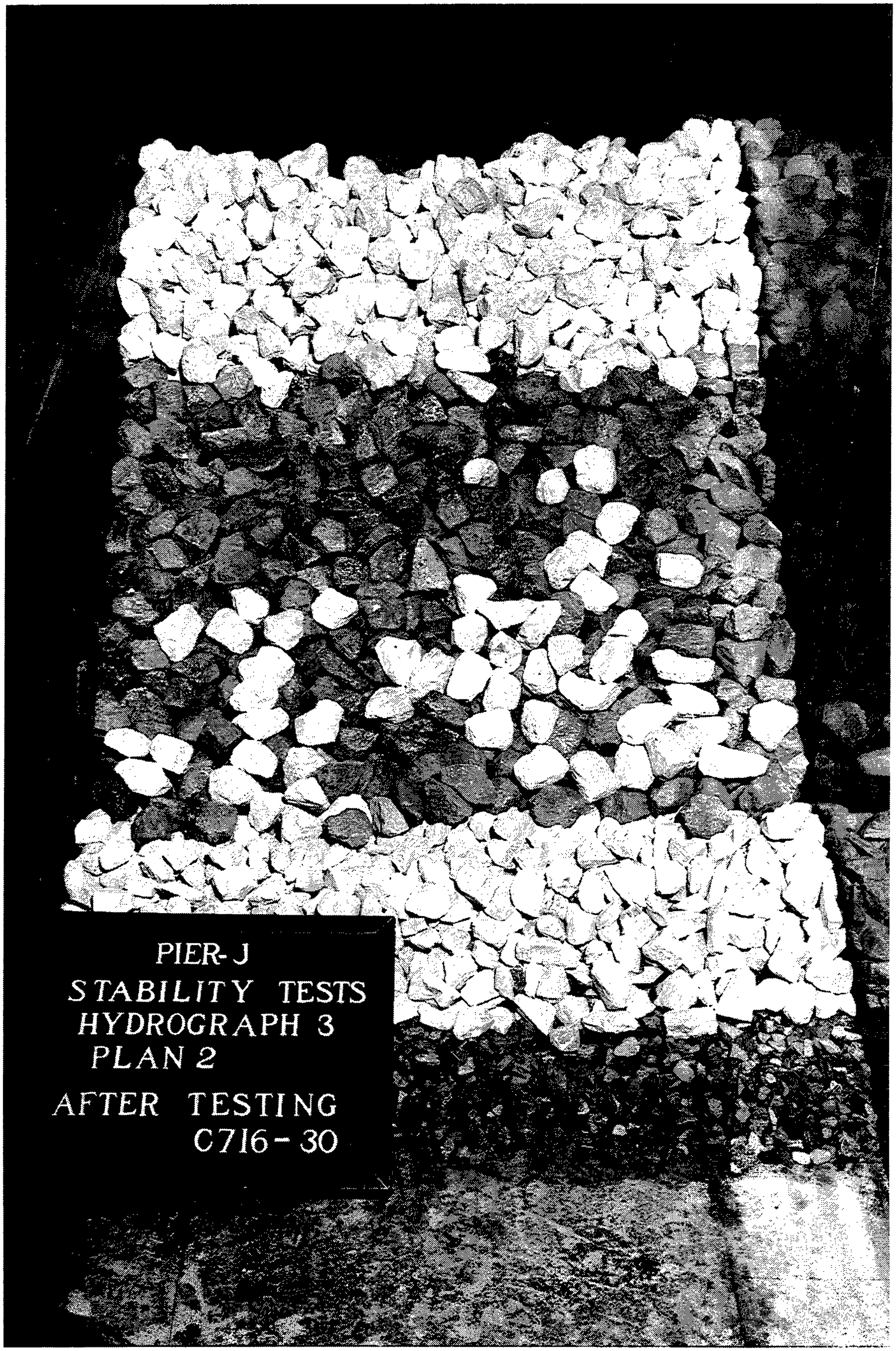


Photo 23. Sea-side view of Plan 2 at the conclusion of testing

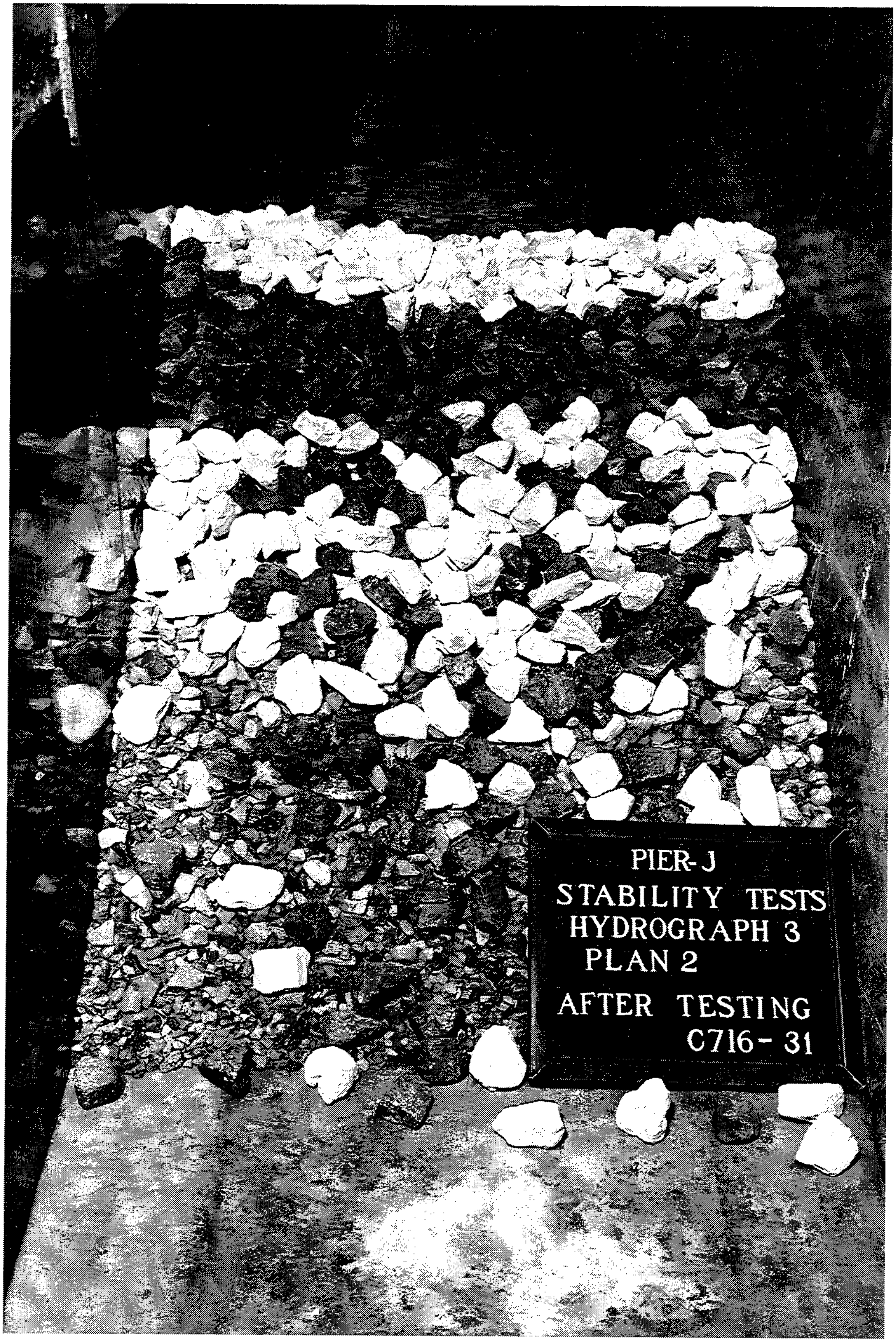


Photo 24. Harbor-side view of Plan 2 at the conclusion of testing

Photo 25. End view of Plan 3 before wave attack

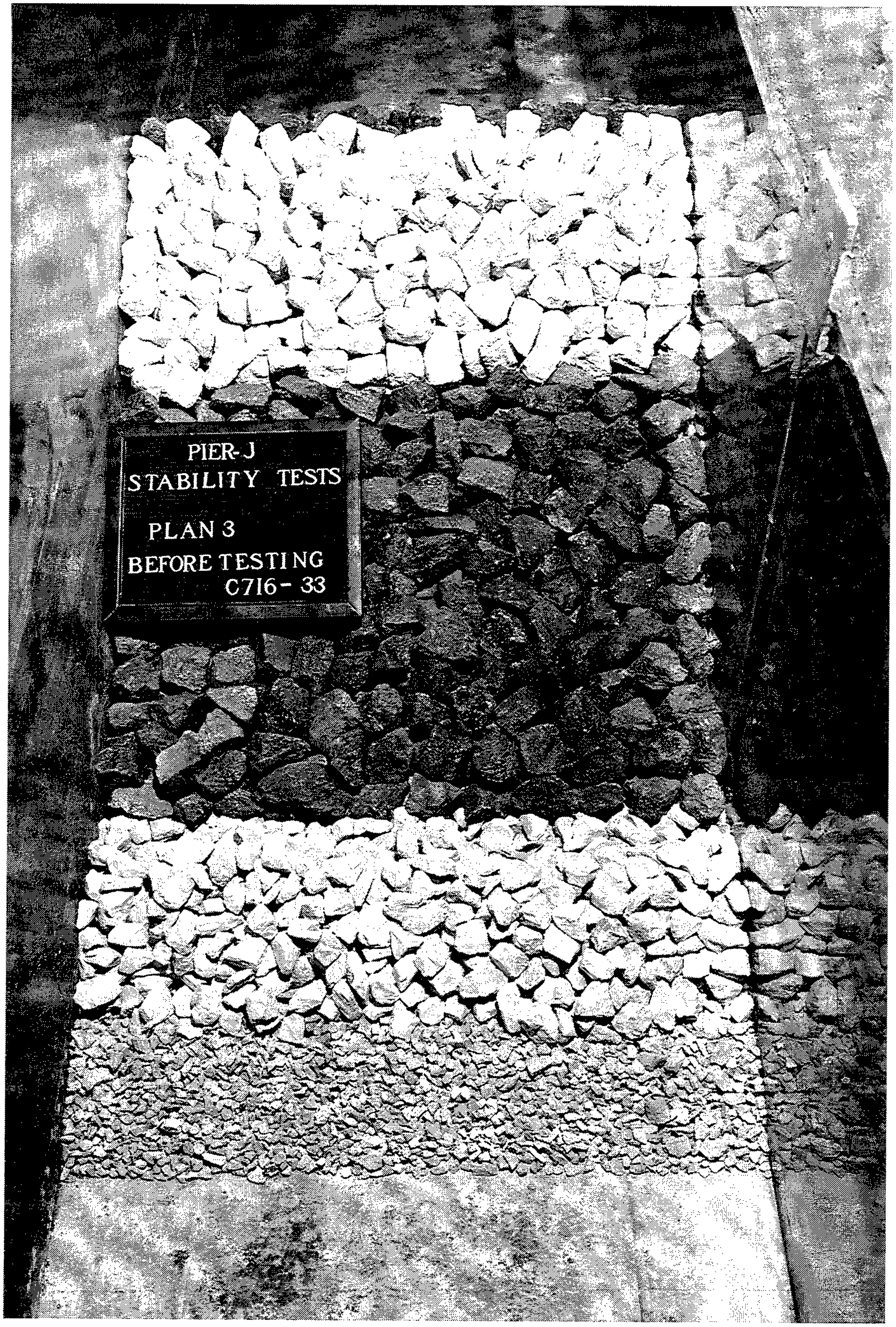


Photo 26. Sea-side view of Plan 3 before wave attack

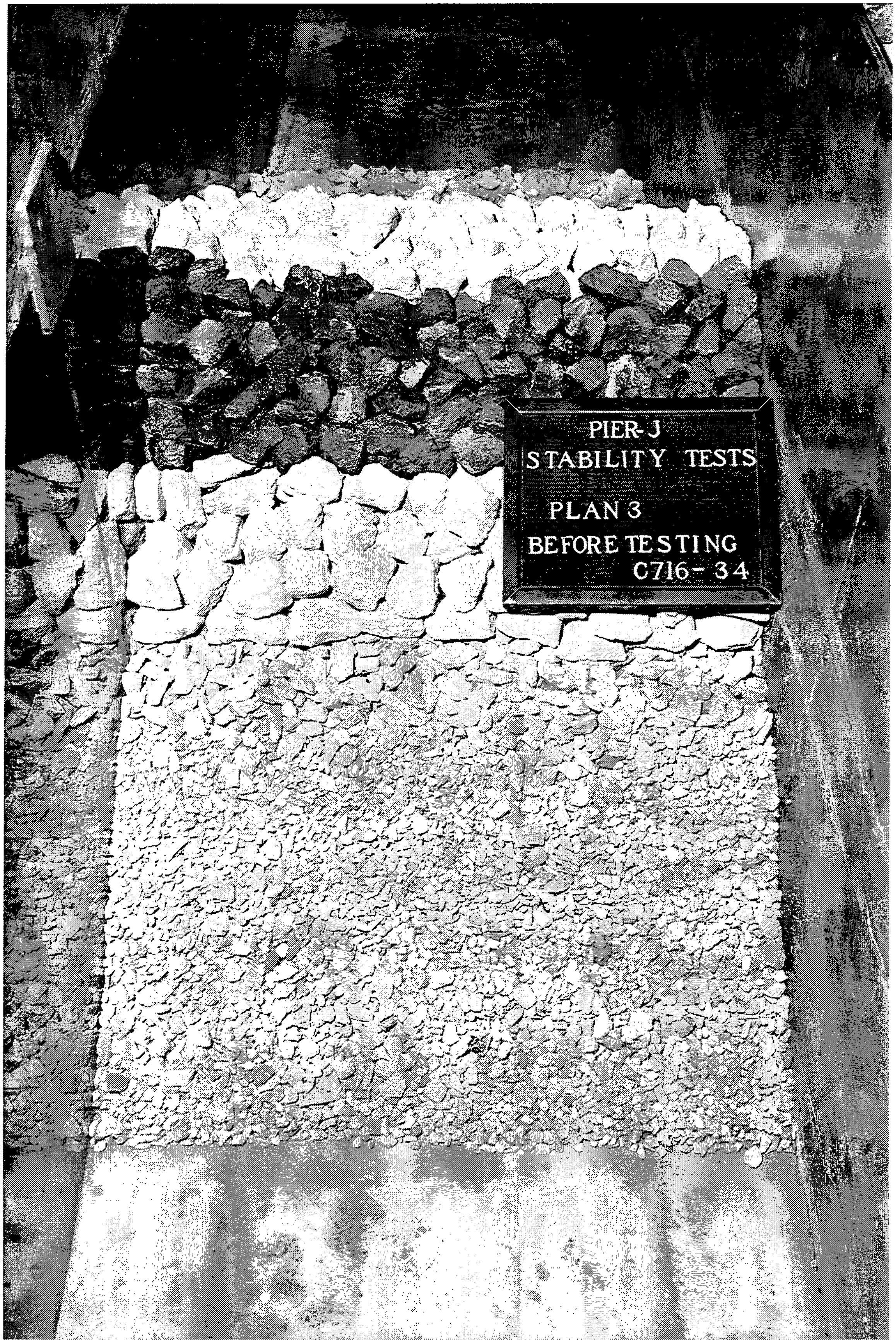
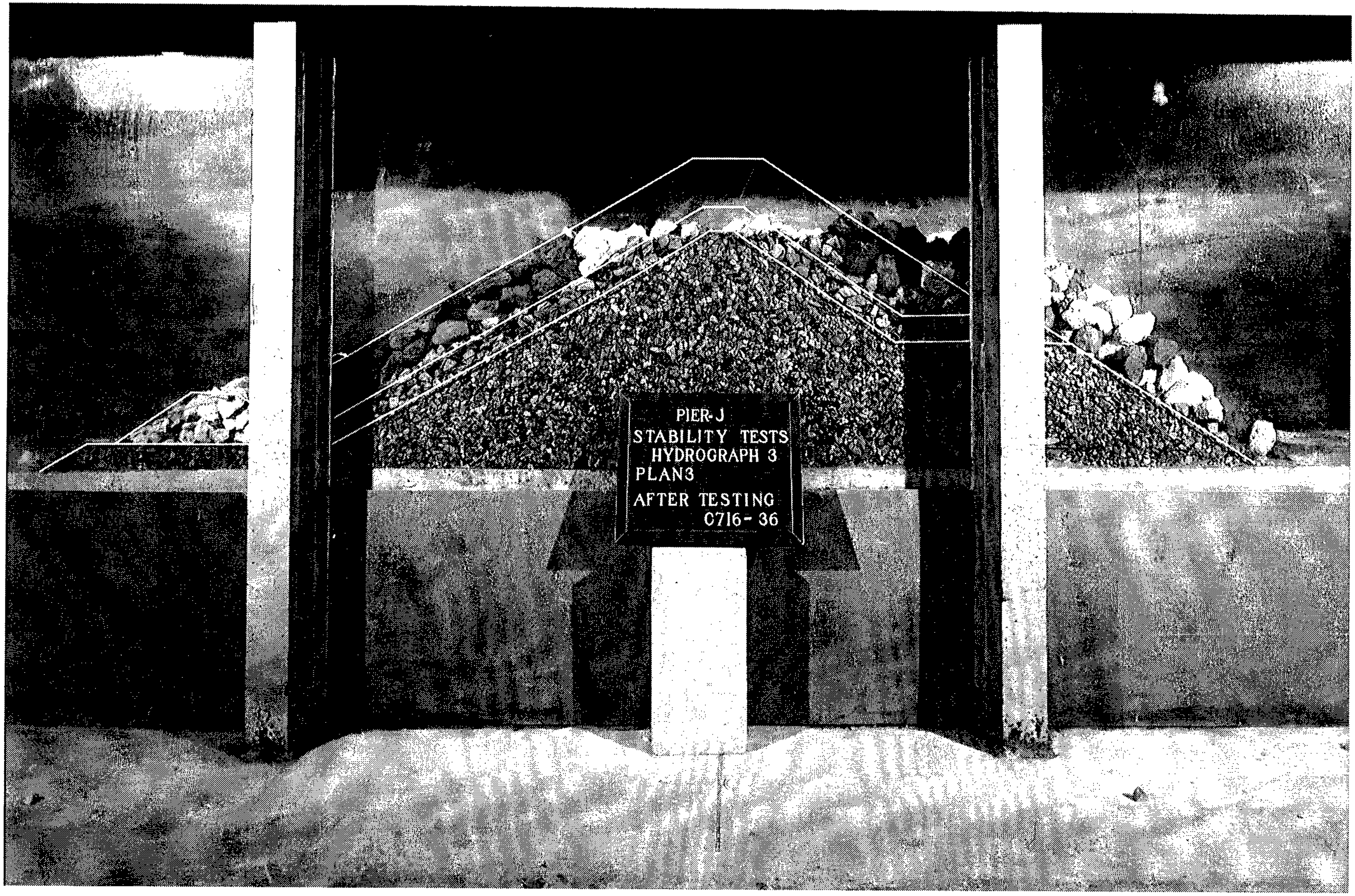


Photo 27. Harbor-side view of Plan 3 before wave attack



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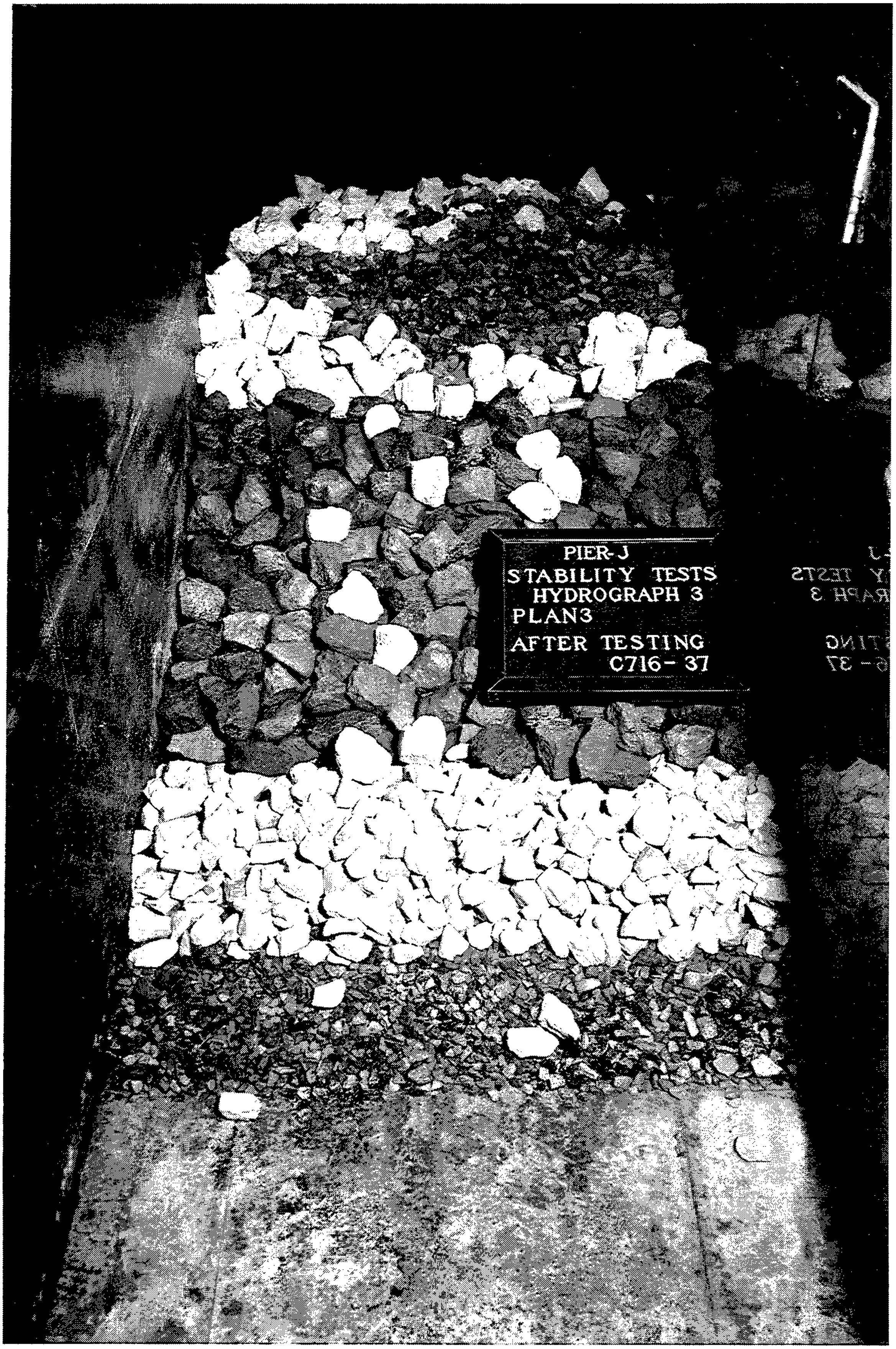
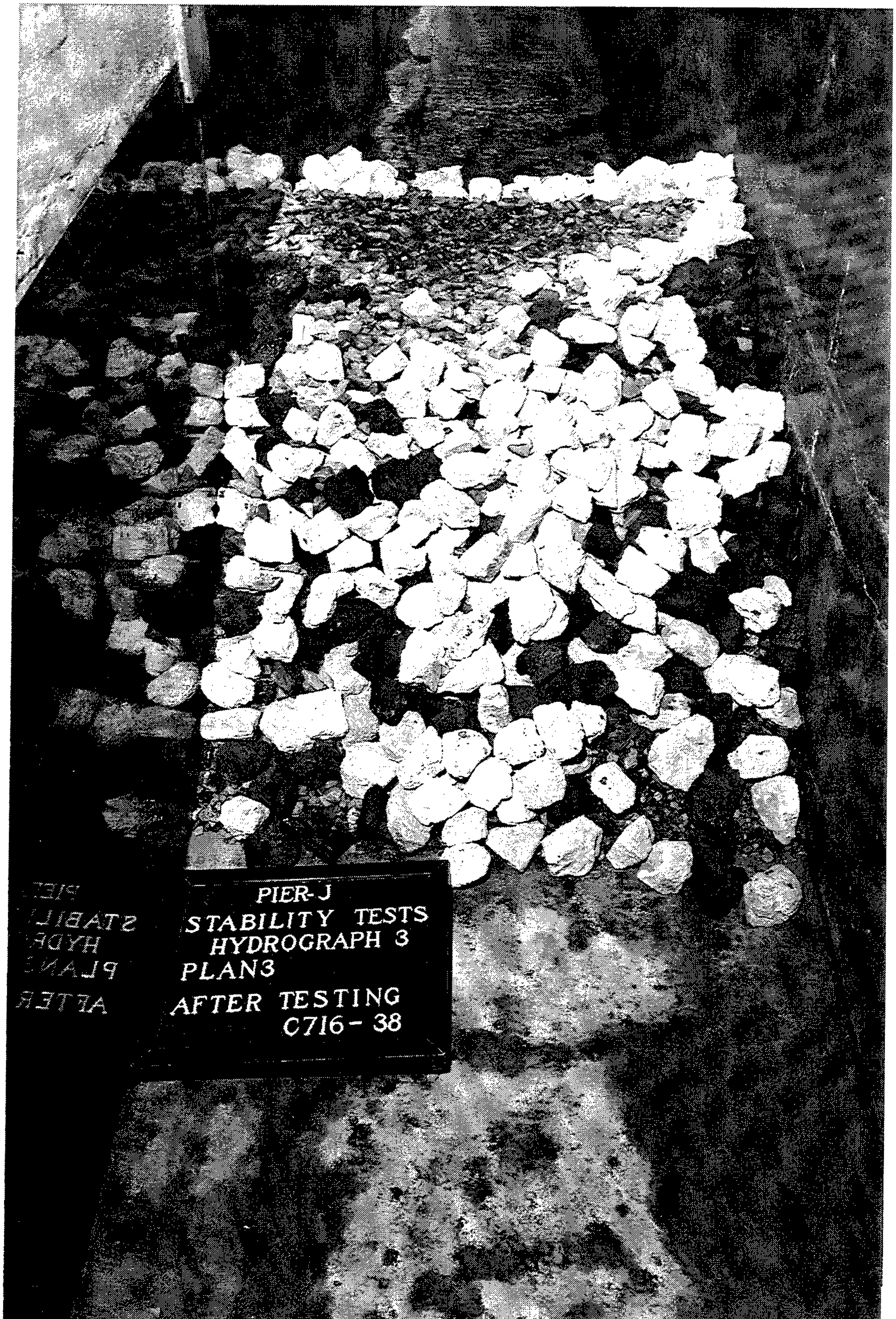


Photo 29. Sea-side view of Plan 3 at the conclusion of testing



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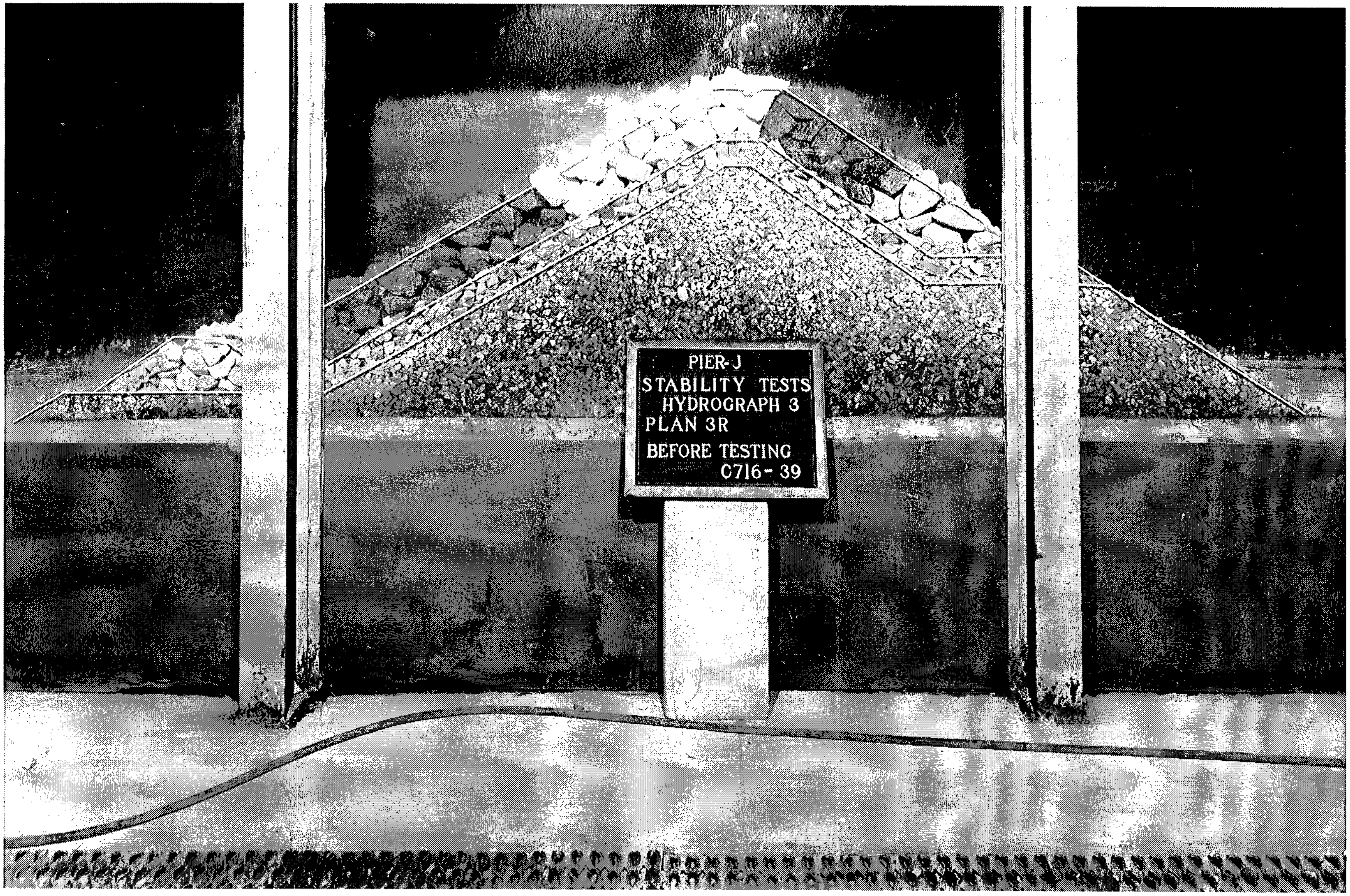


Photo 31. End view of Plan 3R before wave attack

Photo 32. Sea-side view of Plan 3R before wave attack

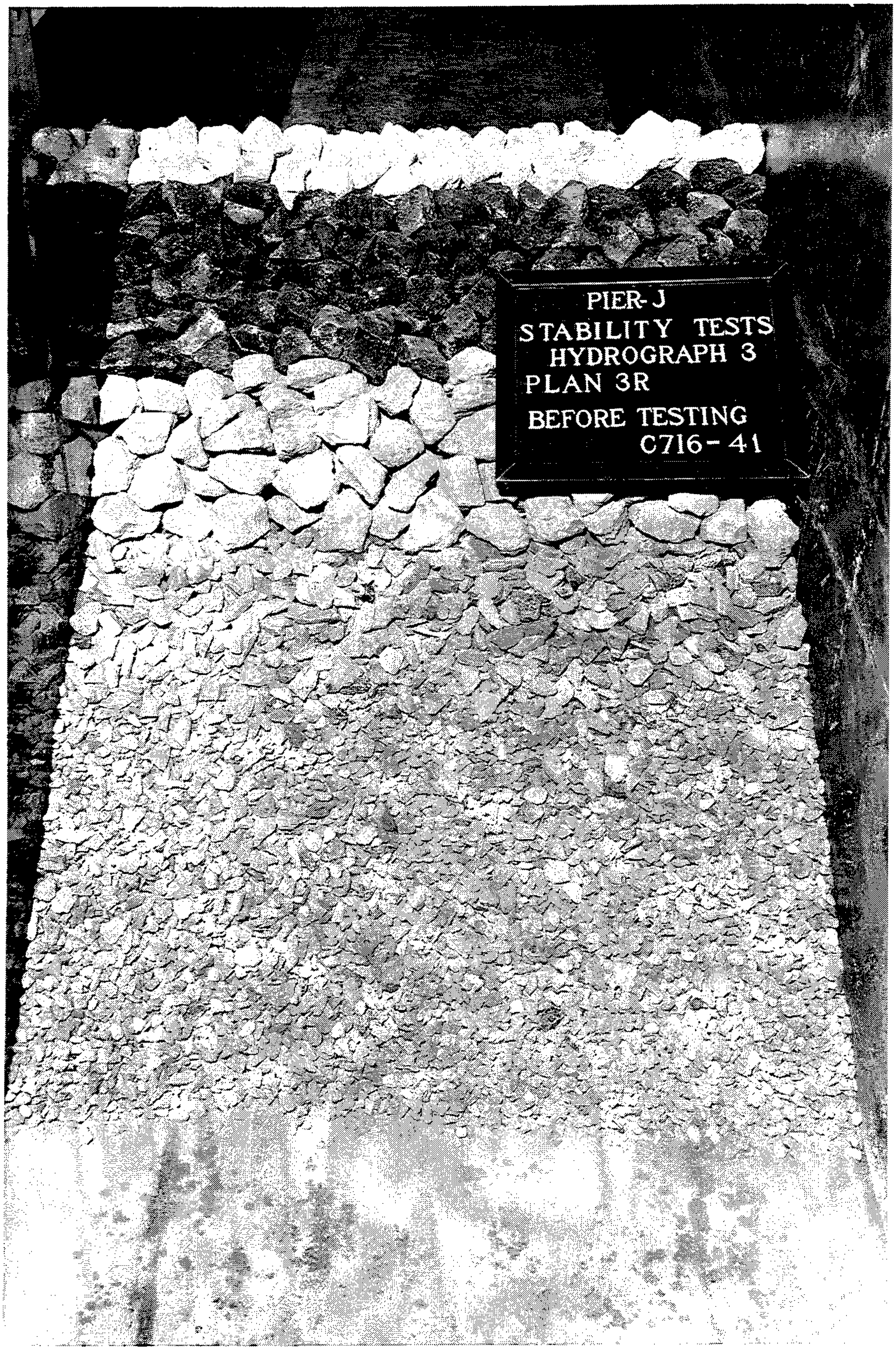


Photo 33. Harbor-side view of Plan 3R before wave attack

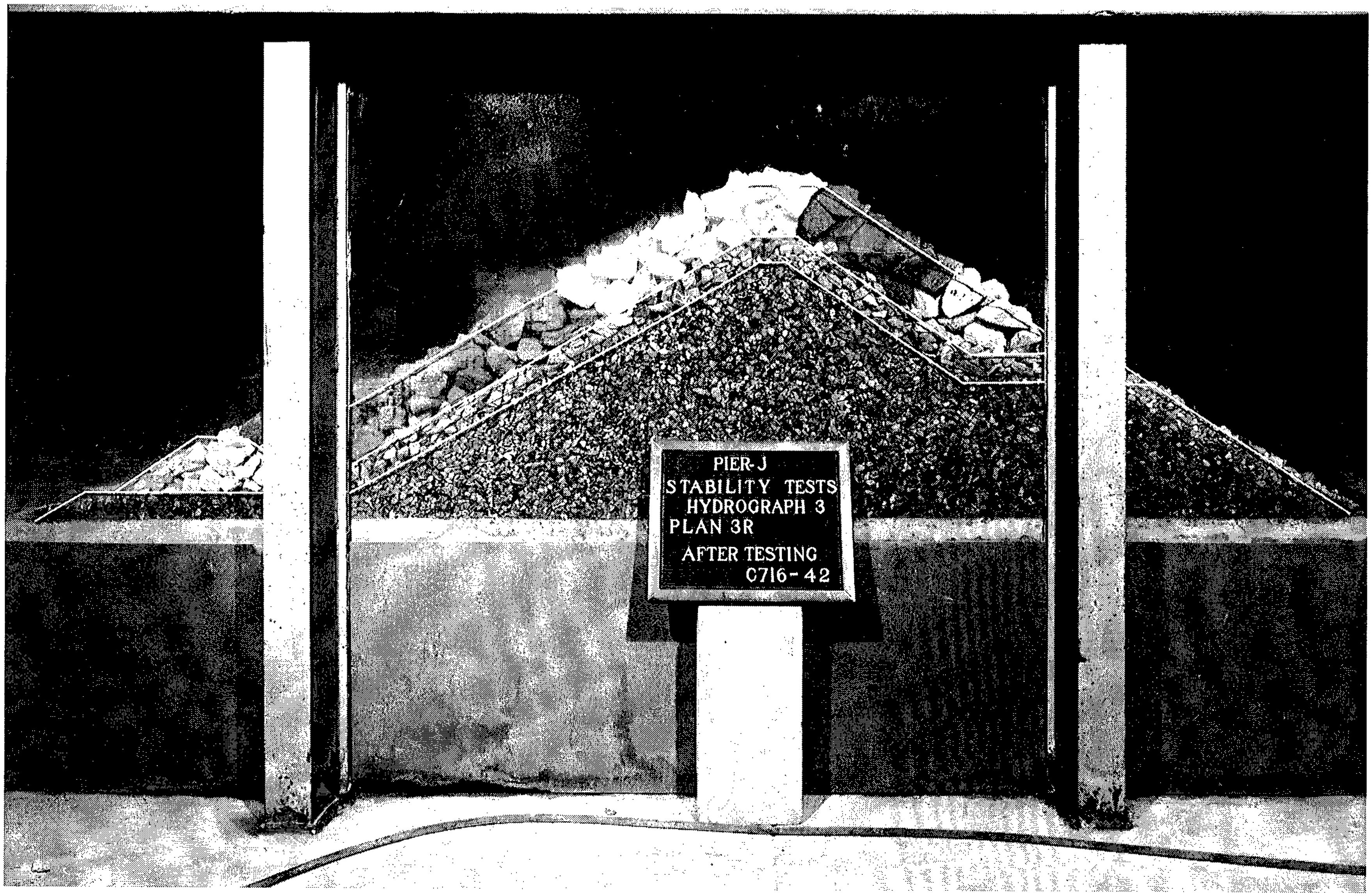


Photo 34. End view of Plan 3R at the conclusion of testing

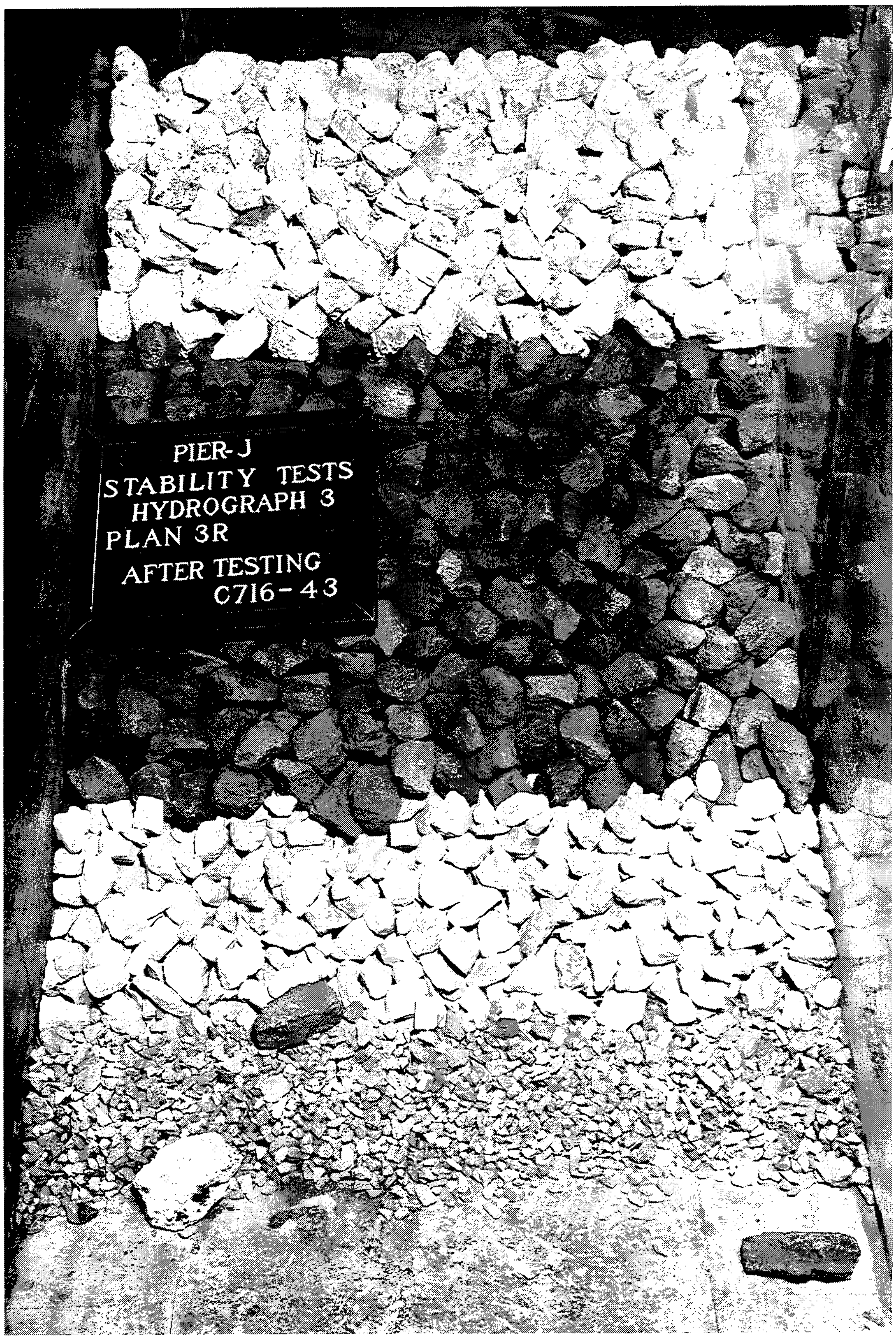
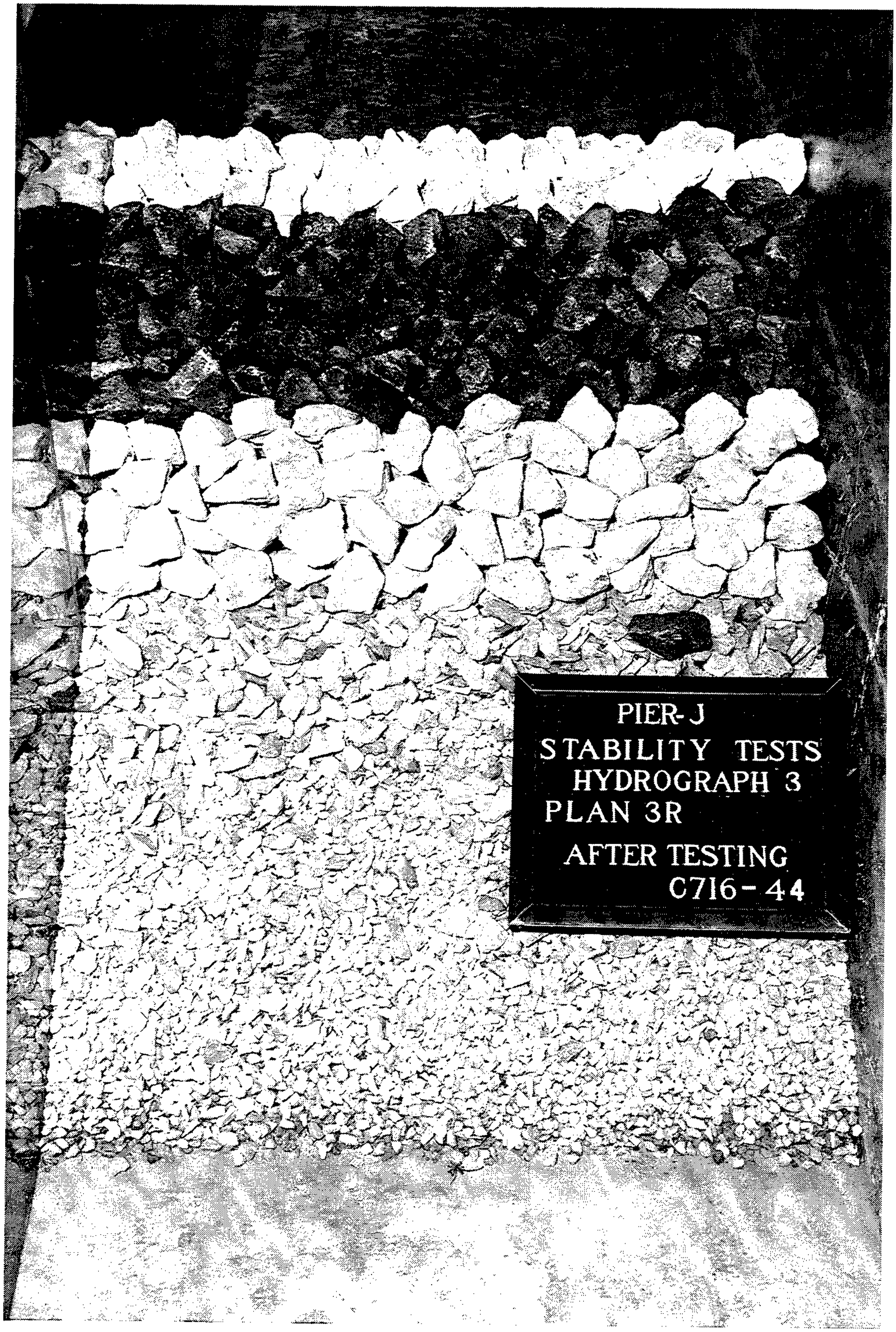


Photo 35. Sea-side view of Plan 3R at the conclusion of testing



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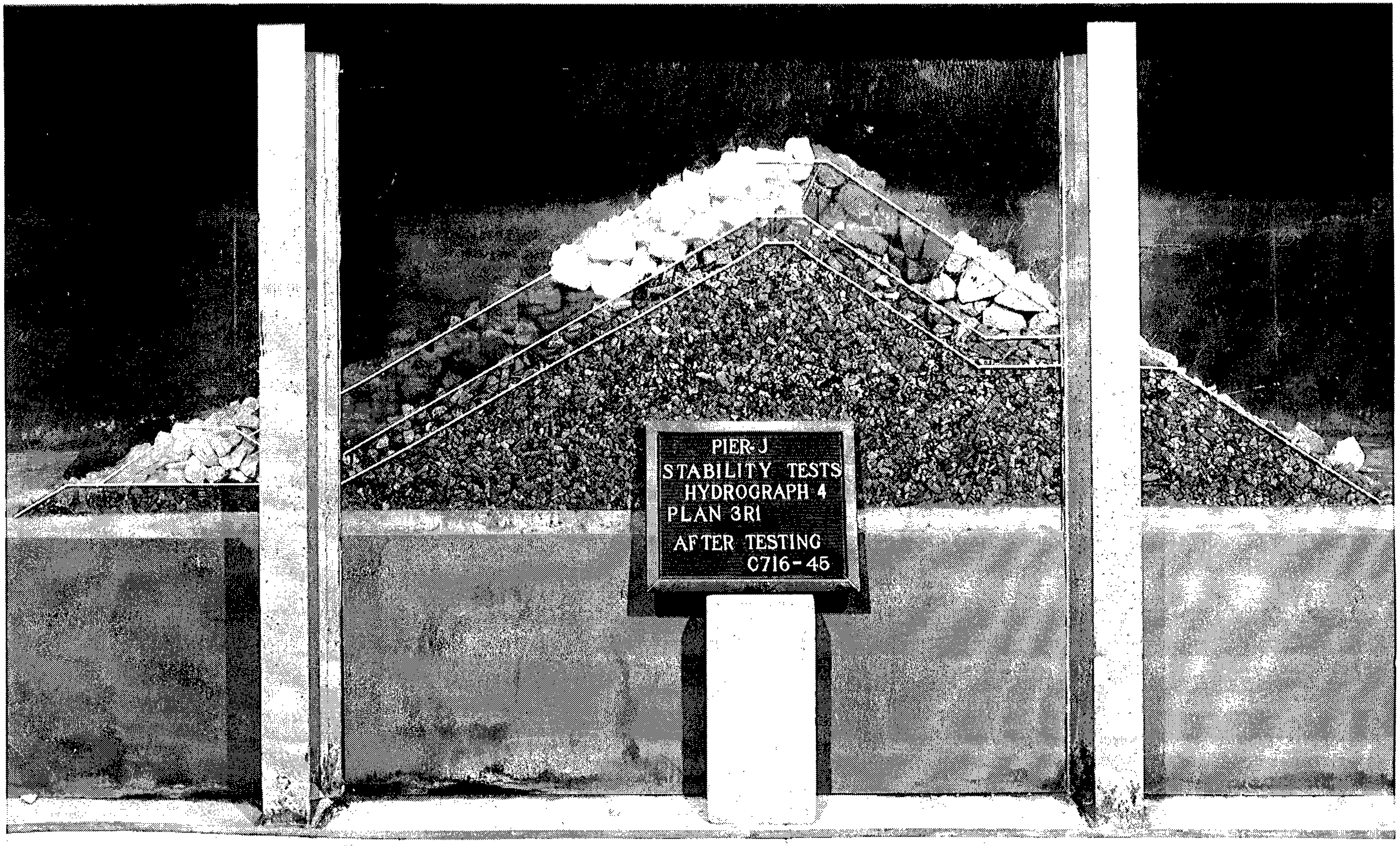

Photo 37. End view of Plan 3R1 at the conclusion of testing

Photo 38. Sea-side view of Plan 3R1 at the conclusion of testing

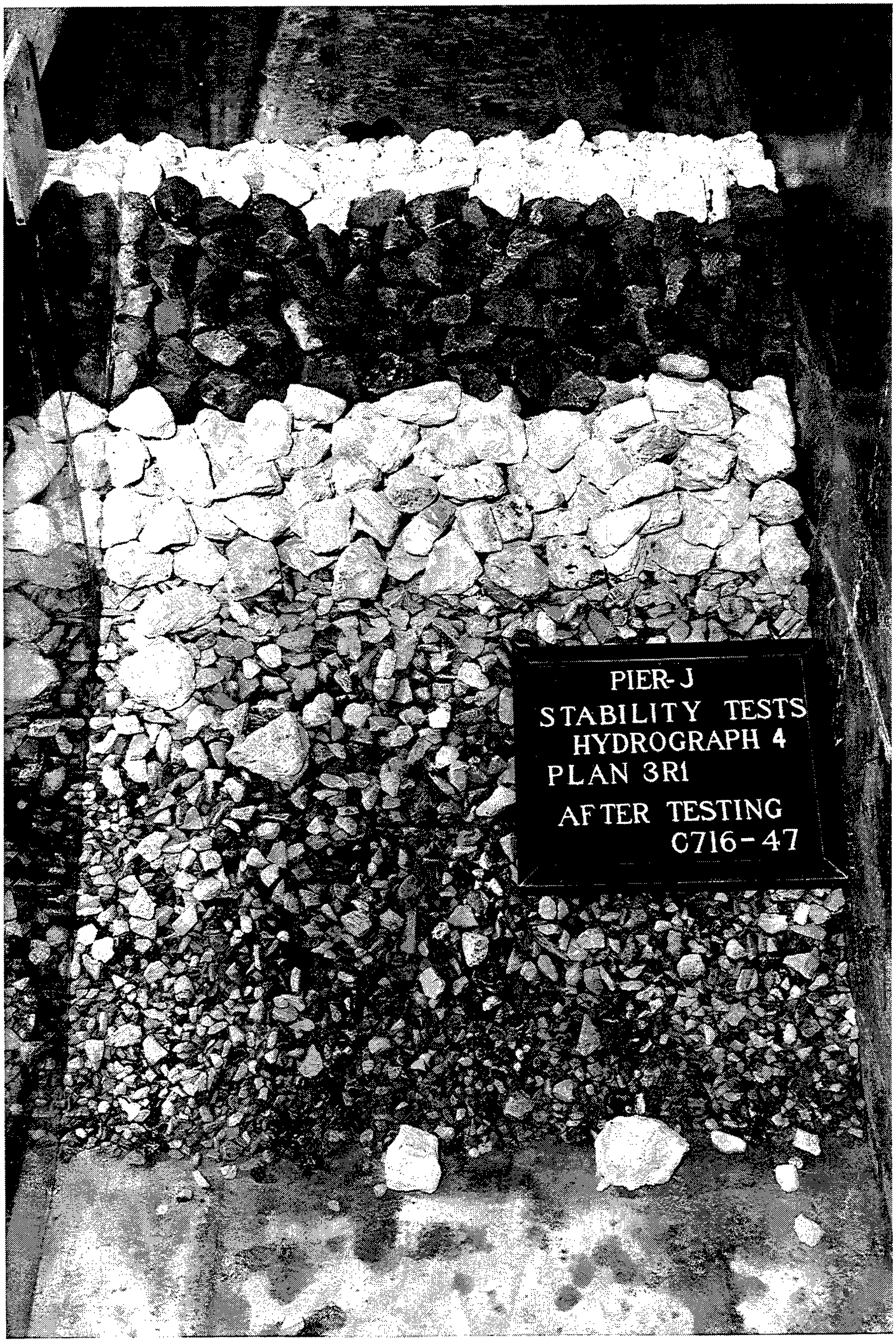

Photo 39. Harbor-side view of Plan 3R1 at the conclusion of testing

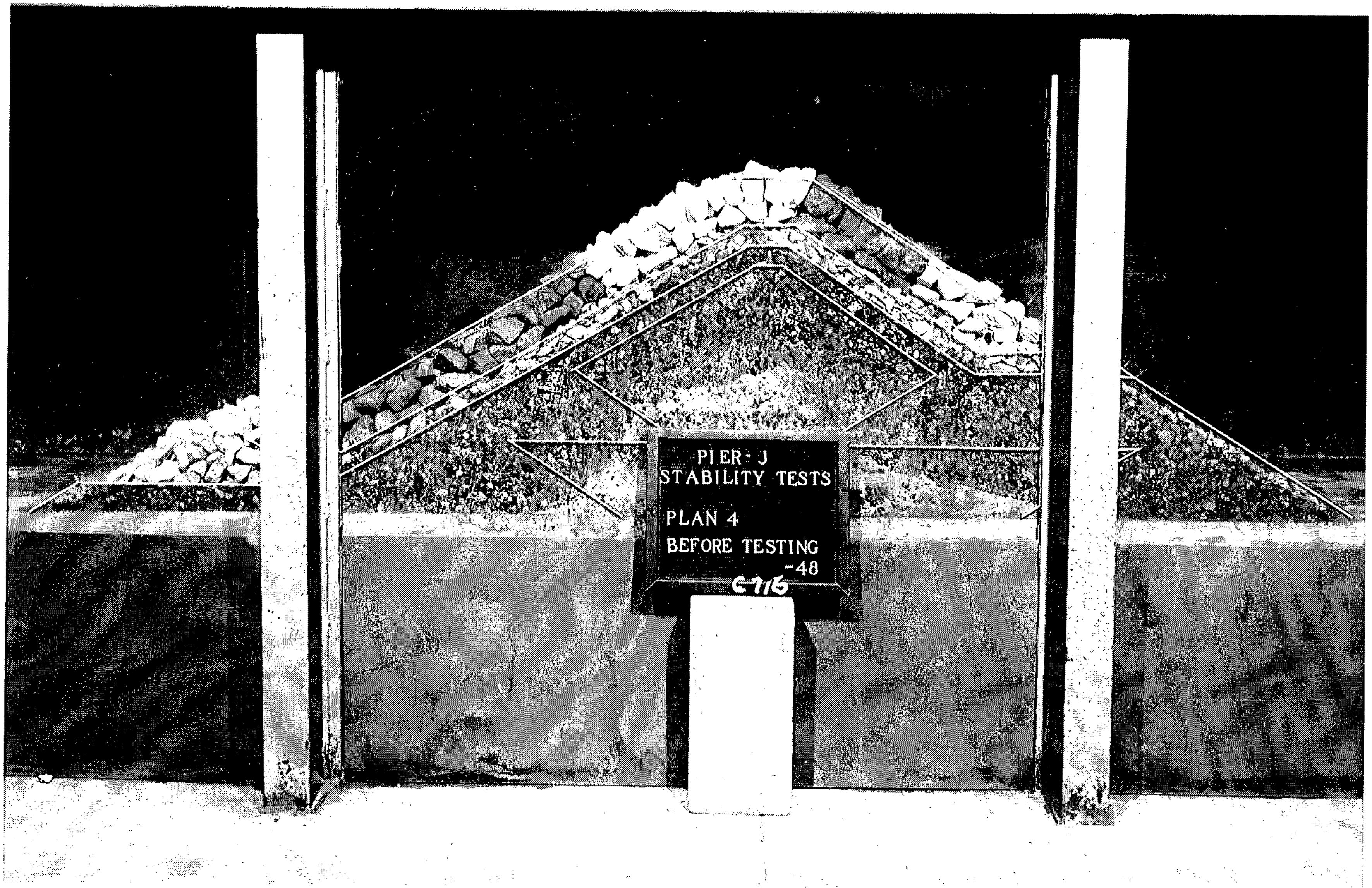

Photo 40. End view of Plan 4 before wave attack

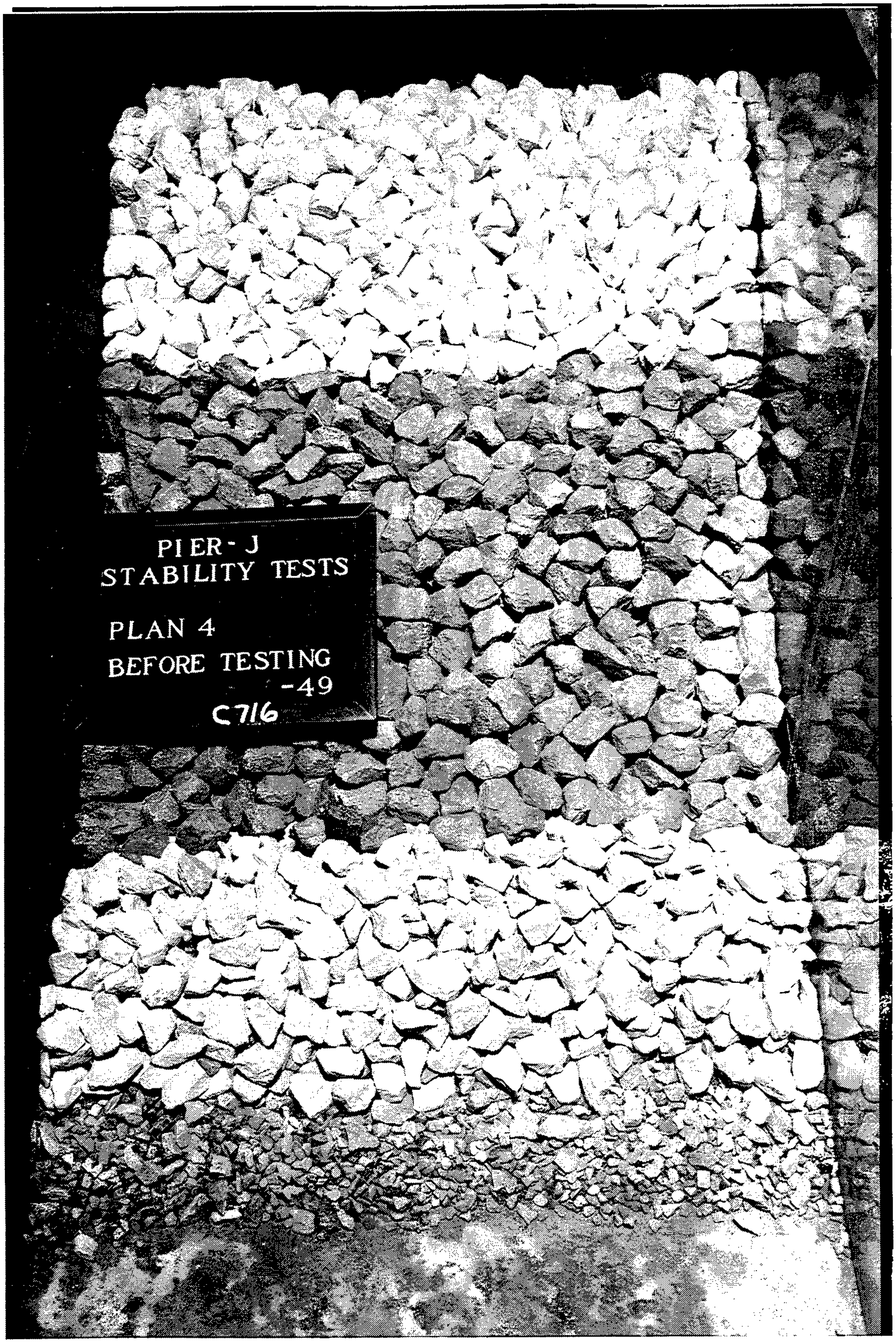

Photo 41. Sea-side view of Plan 4 before wave attack

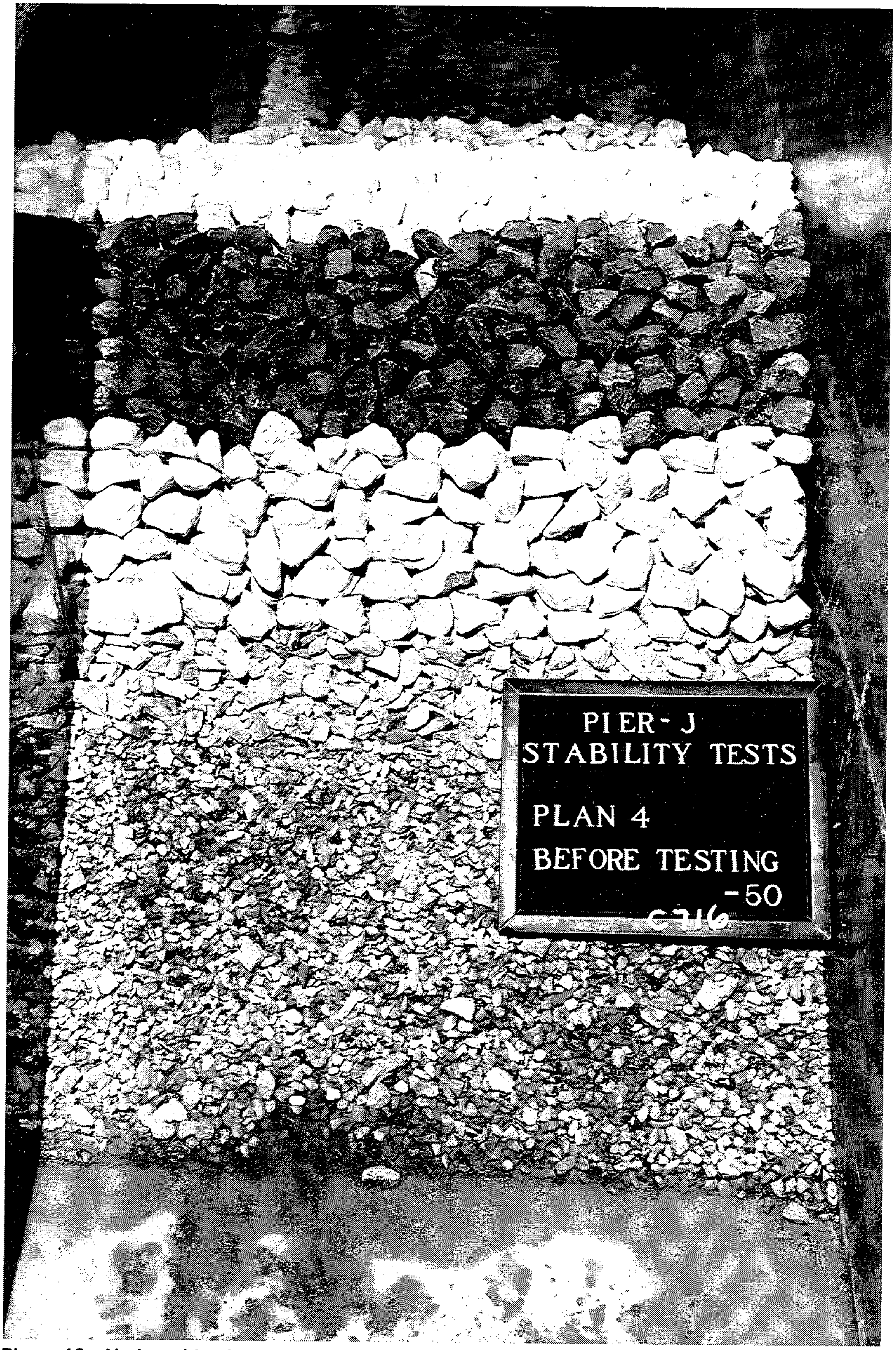

Photo 42. Harbor-side view of Plan 4 before wave attack

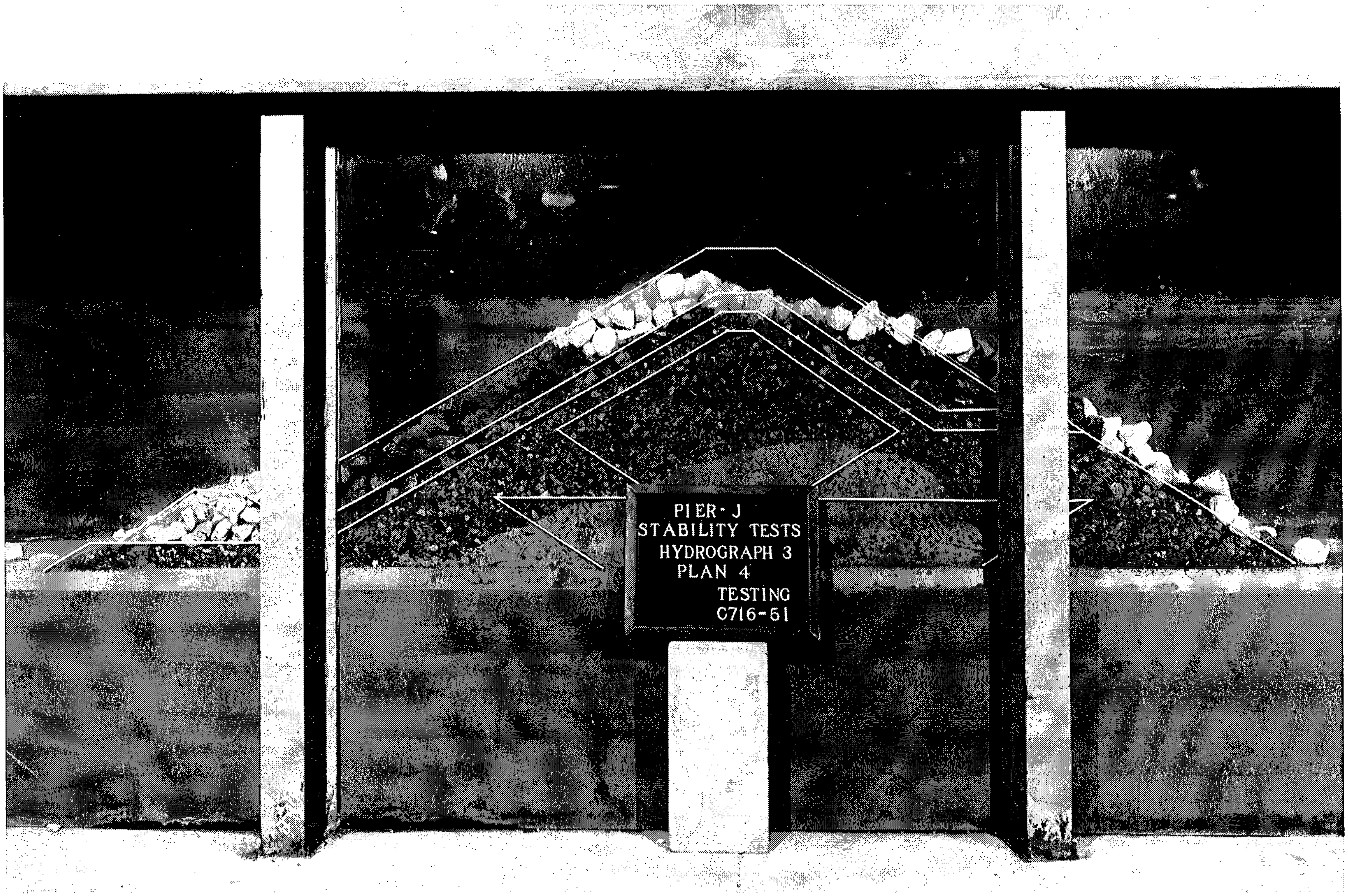

Photo 43. End view of Plan 4 after testing step 15 of Hydrograph 3

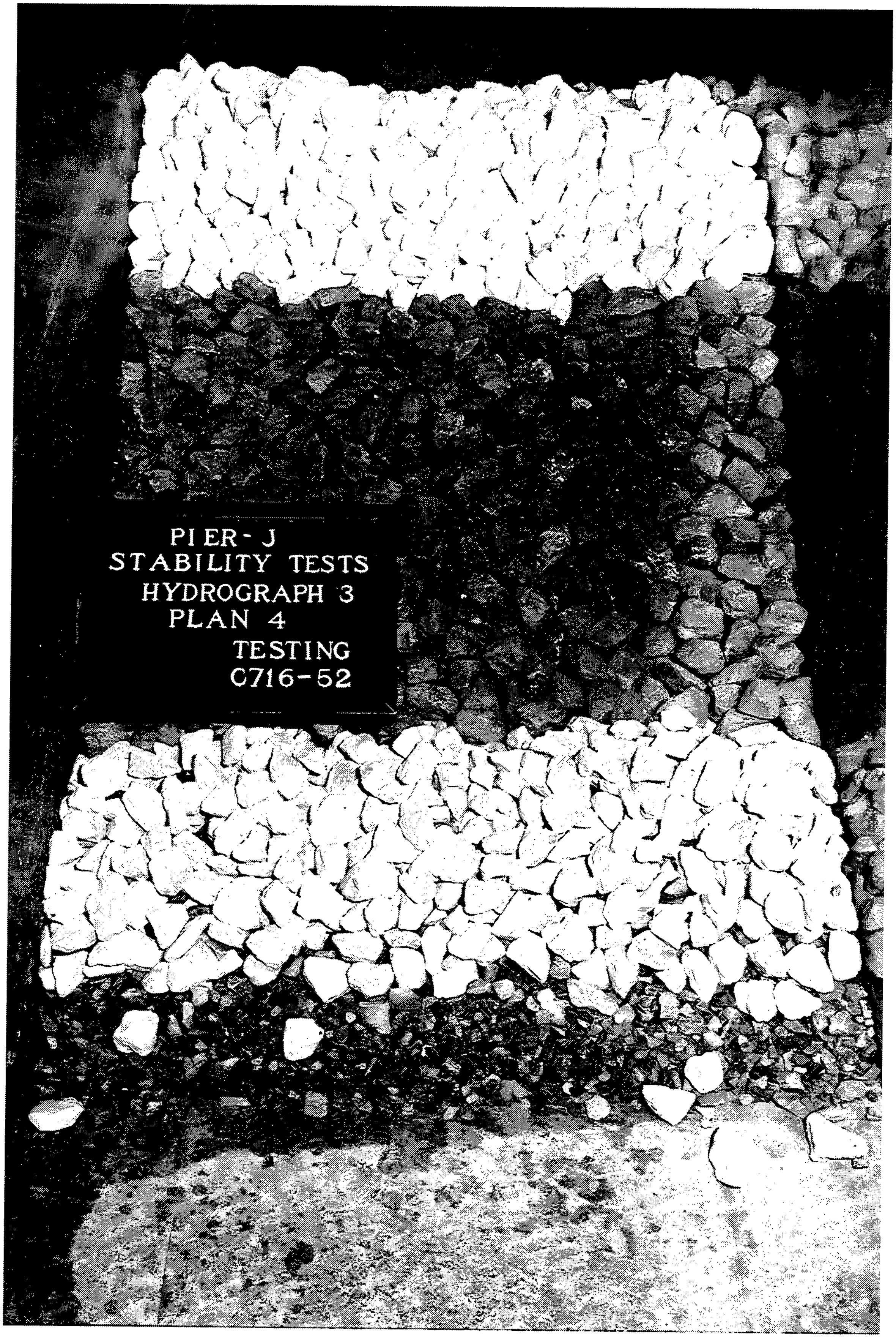

Photo 44. Sea-side view of Plan 4 after testing step 15 of Hydrograph 3

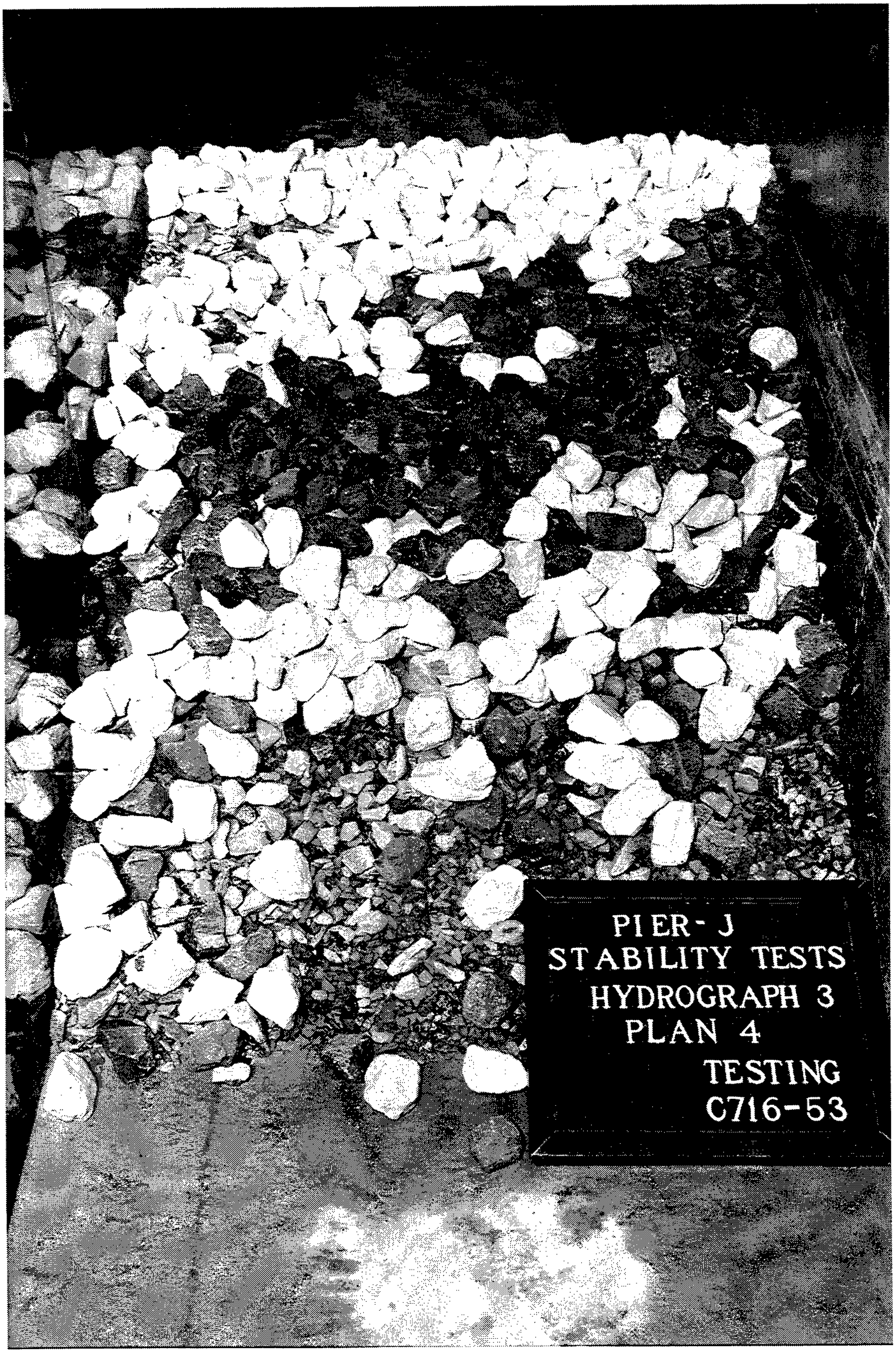

Photo 45. Harbor-side view of Plan 4 after testing step 15 of Hydrograph 3

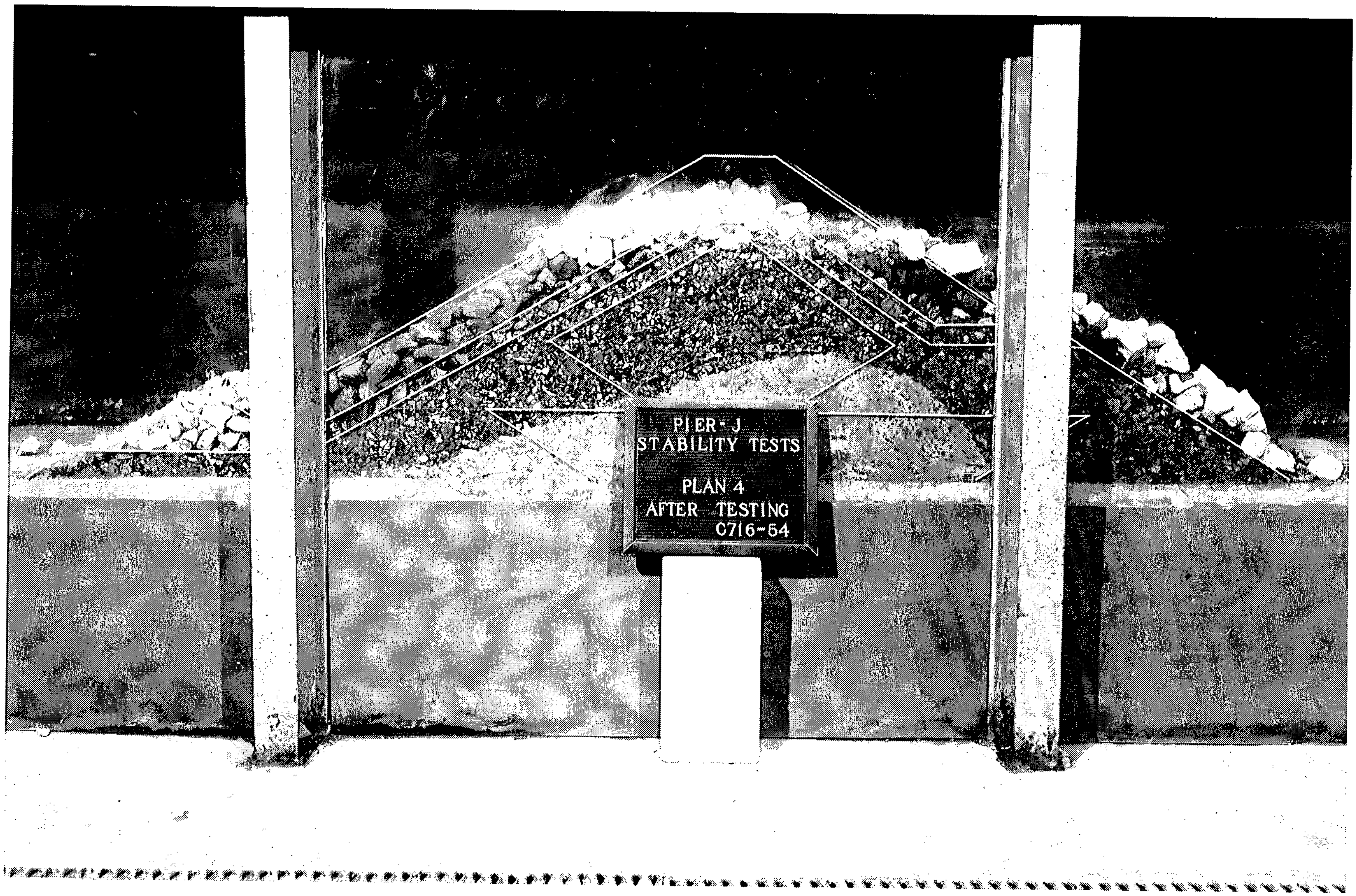

Photo 46. End view of Plan 4 at the conclusion of testing

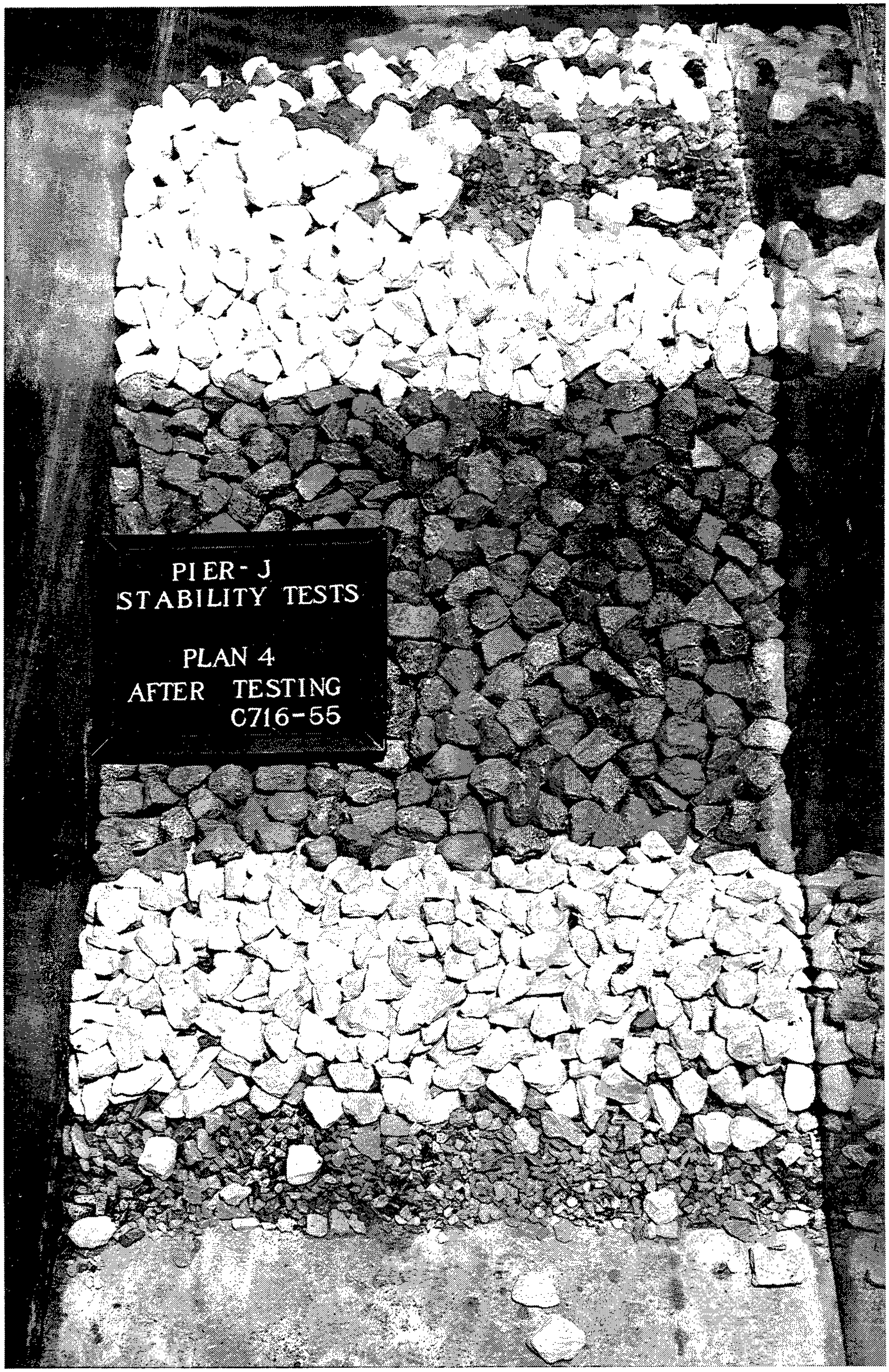

Photo 47. Sea-side view of Plan 4 at the conclusion of testing

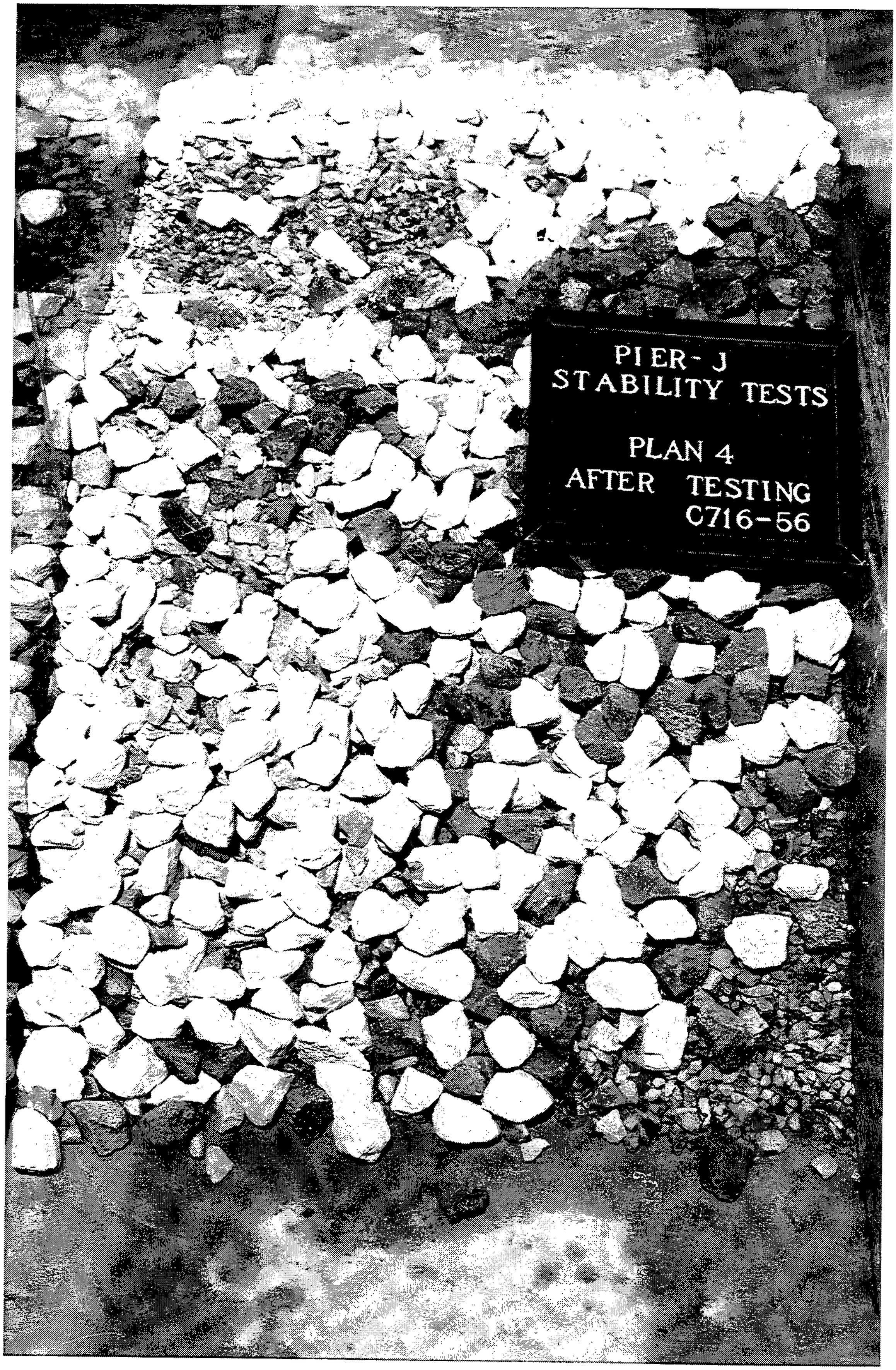

Photo 48. Harbor-side view of Plan 4 at the conclusion of testing

Photo 49. Sea-side view of Plan 4R after testing step 12 of Hydrograph 3

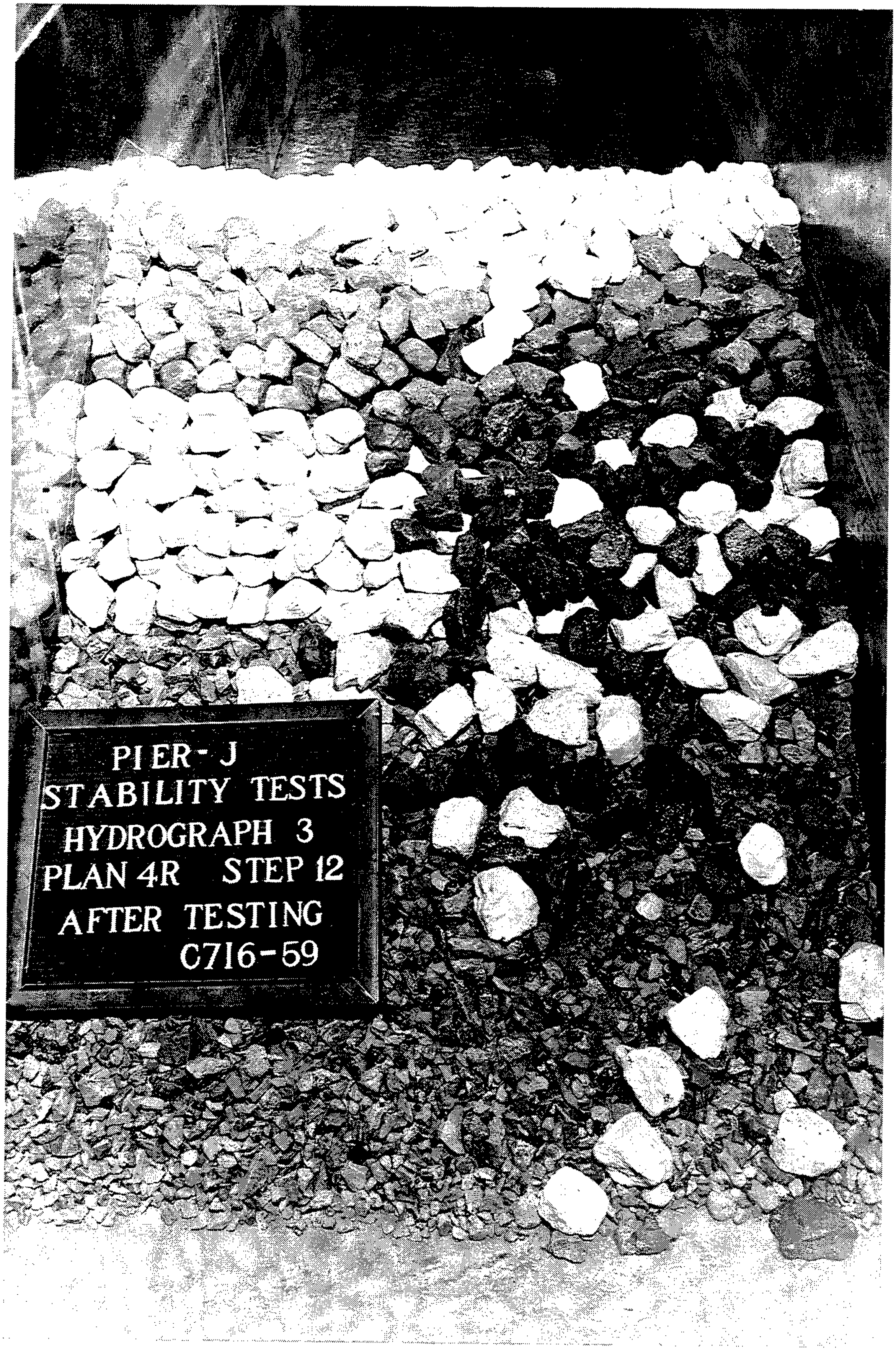
PIER-J
STABILITY TESTS
HYDROGRAPH 3
PLAN 4R STEP 12
AFTER TESTING
C716-59

Photo 51. End view of Plan 4R after testing step 15 of Hydrograph 3

Photo 52. Harbor-side view of Plan 4R after testing step 15 of Hydrograph 3

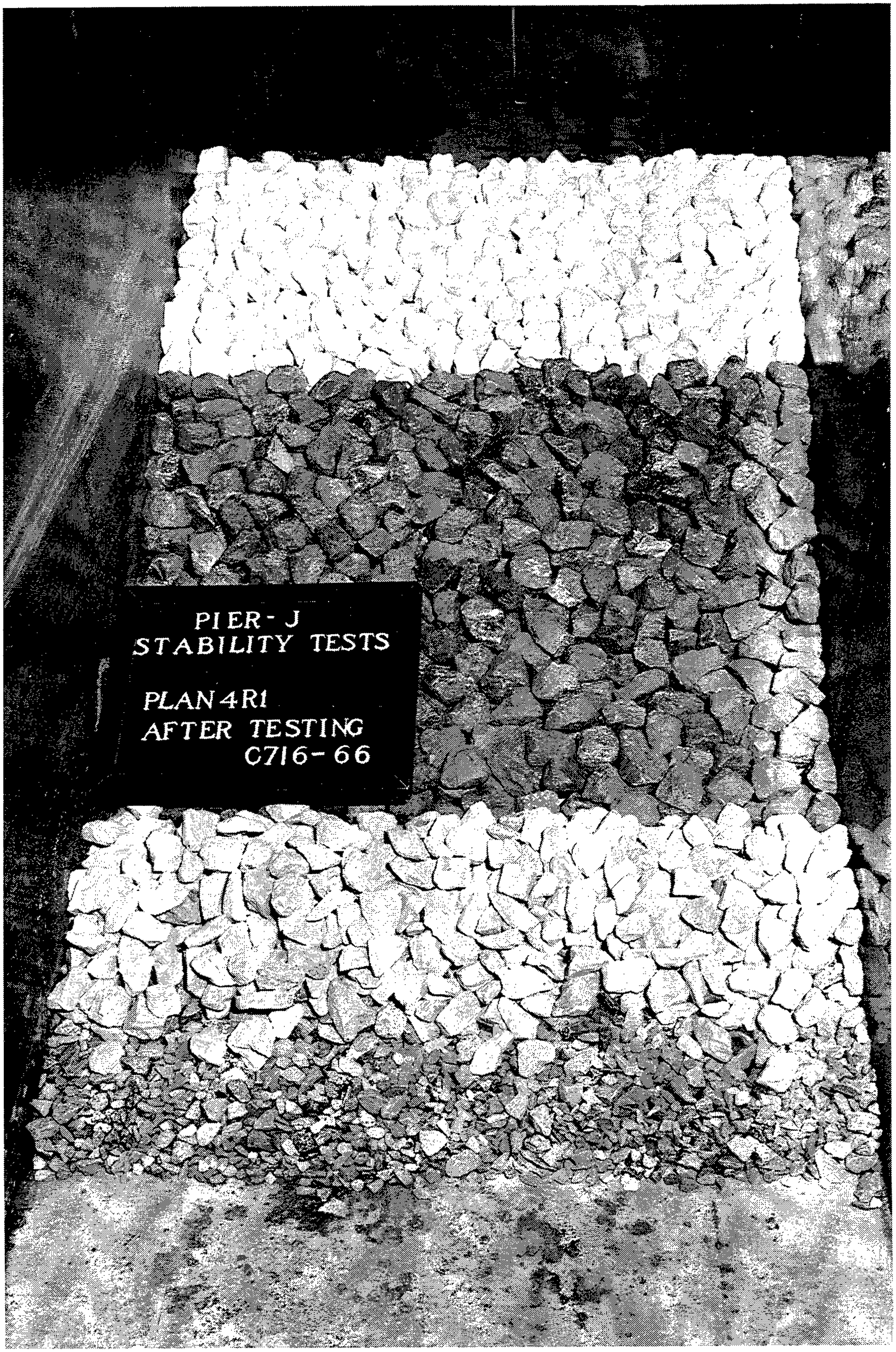

Photo 53. Sea-side view of Plan 4R1 after testing step 14 of Hydrograph 3A

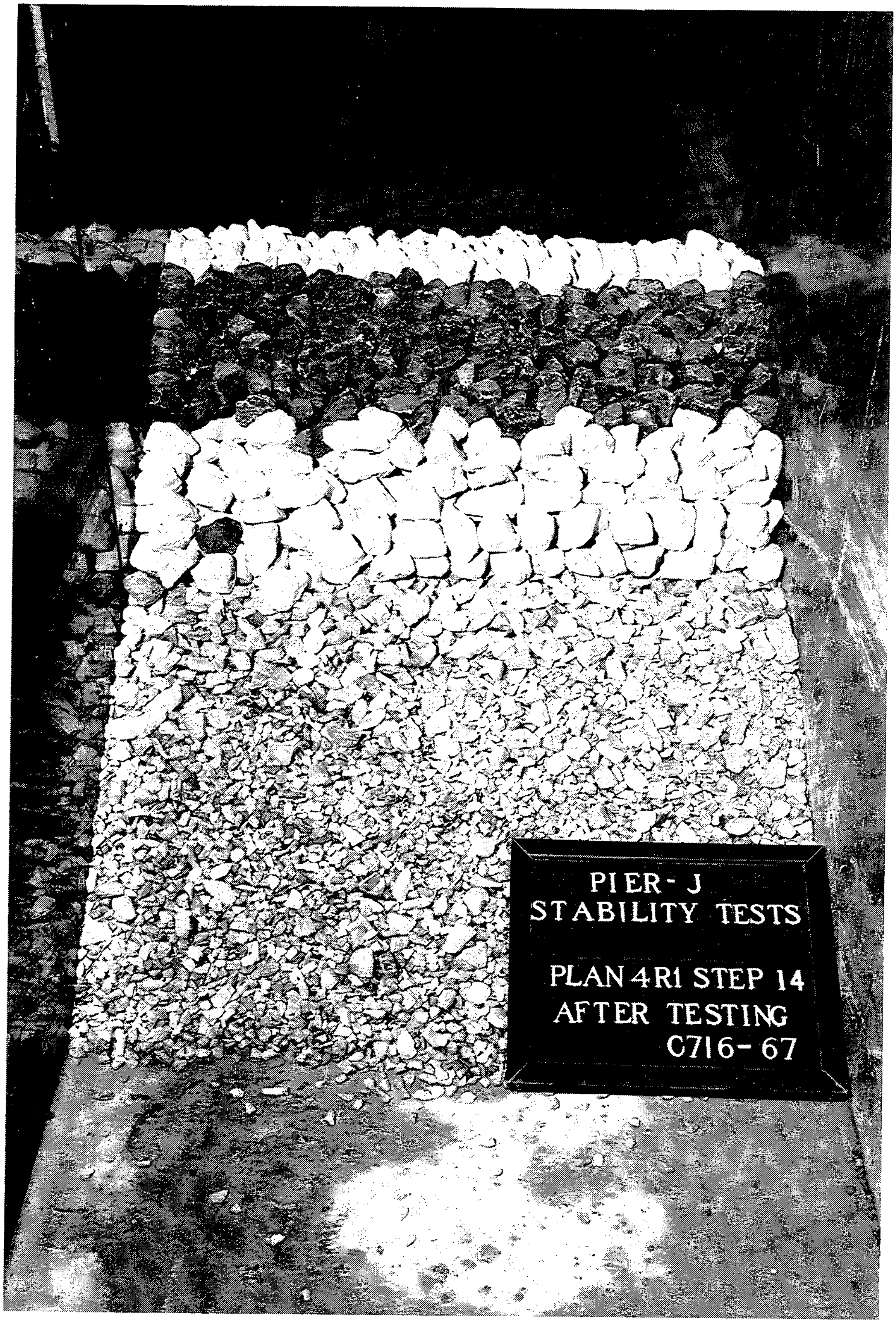

Photo 54. Harbor-side view of Plan 4R1 after testing step 14 of Hydrograph 3A

Photo 55. Sea-side view of Plan 4R1 after testing step 18 of Hydrograph 3A

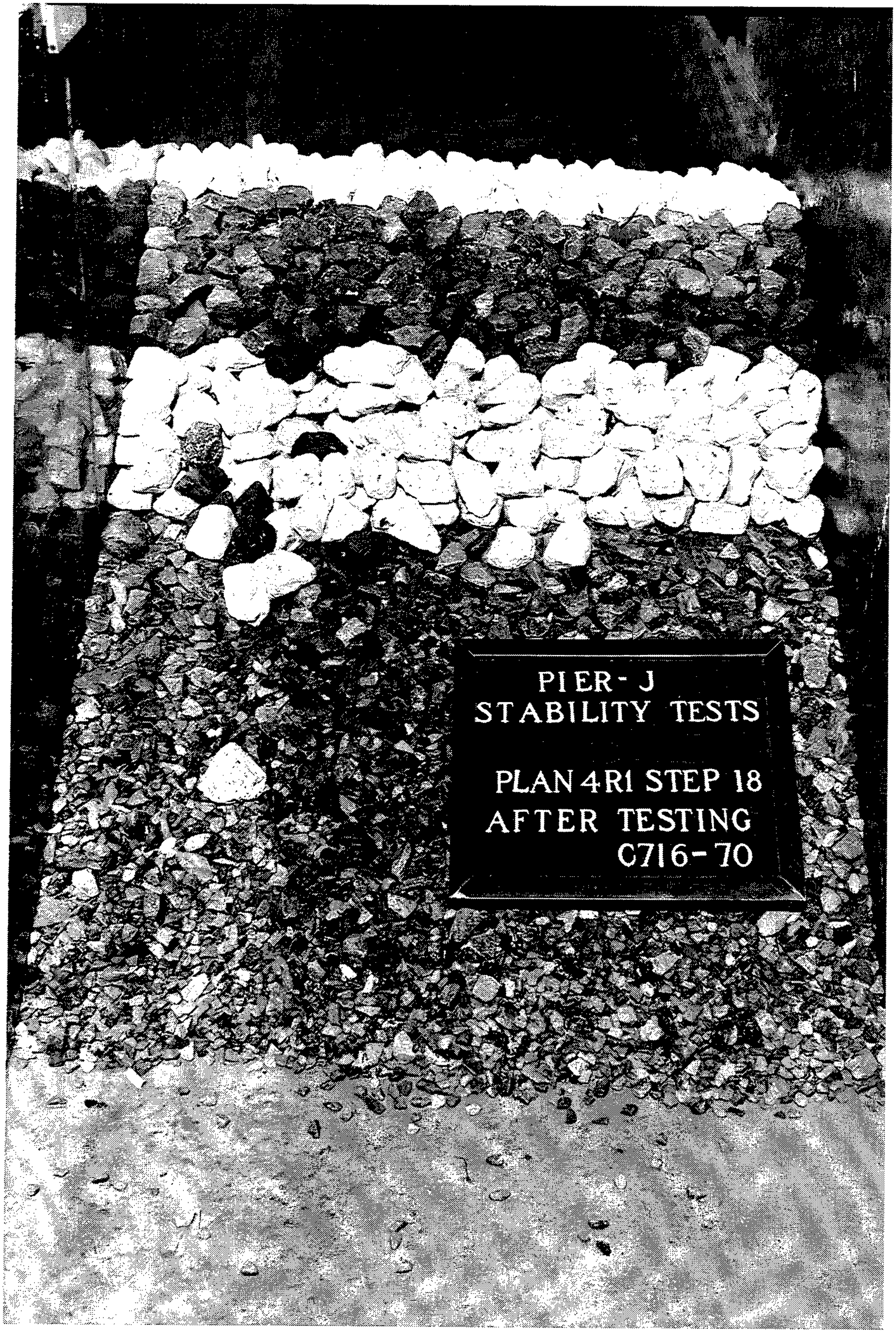

Photo 56. Harbor-side view of Plan 4R1 after testing step 18 of Hydrograph 3A

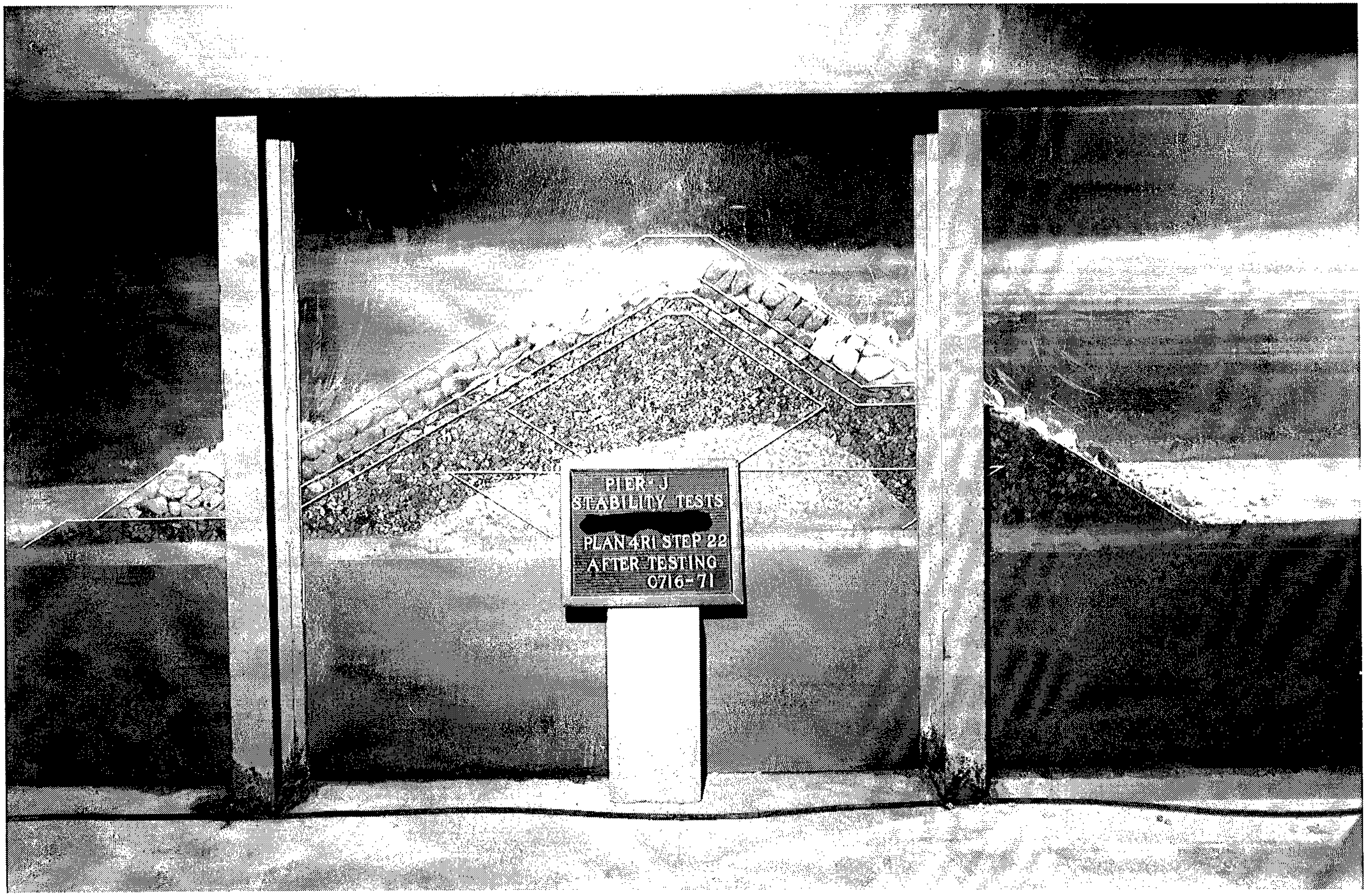

Photo 57. End view of Plan 4R1 at the conclusion of testing

Photo 58. Sea-side view of Plan 4R1 at the conclusion of testing

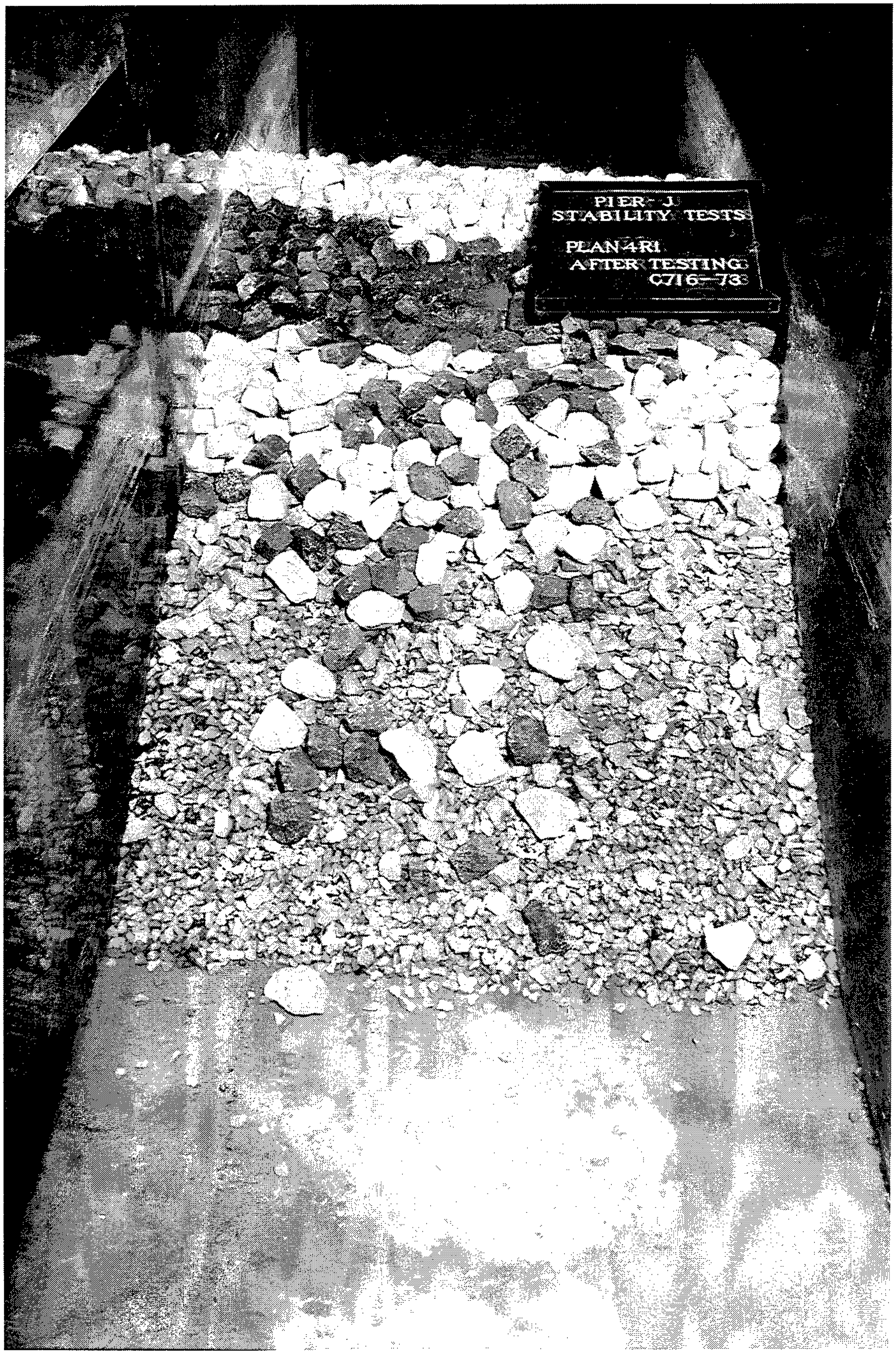

Photo 59. Harbor-side view of Plan 4R1 at the conclusion of testing

Appendix A
Notation

H_{mo}	Zero-moment wave height, ft
L	Length
L_p	Airy wave length, ft
L^2	Area
L^3	Volume
M	Model quantity
P	Prototype quantity
R_N	Reynolds stability number
T	Time
T_p	Wave period of peak energy density of spectrum, sec
W_a	Weight, lb
ν_a	Specific weight of armor unit, pcf
L_m/L_p	Linear scale of the model
S_a	Specific gravity of an individual armor unit relative to the water in which it was placed

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12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

Increased commercial activity has created the need for an additional rubble-mound breakwater to protect the Pier J basin of the Port of Long Beach, California. The proposed breakwater will be located on the leeward side of the middle breakwater and will provide for dissipation of long-period wave energy. The investigation described in this report was conducted to determine, by two-dimensional flume tests, the stability response of the proposed breakwater as initially designed, and based on the results of this test, to test additional plans as needed. Transmission of long-period wave energy was also measured for selected plans.

14. SUBJECT TERMS
Long Beach, California
Long-period wave energy
Rubble-mound breakwaters

15. NUMBER OF PAGES
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17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
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